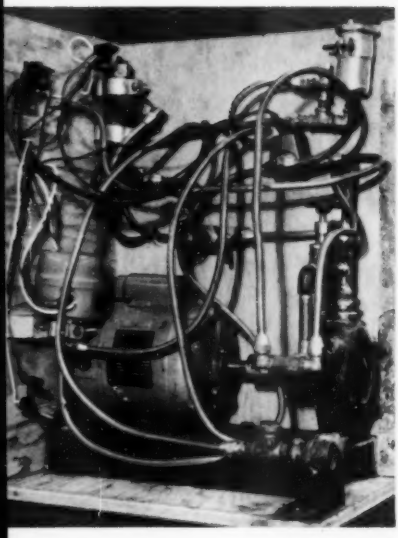
FEBRUARY 1960

Journal

VOL. 52 • NO. 2

AMERICAN WATER WORKS ASSOCIATION



Lactic tactic applied to water—575-gpd pasteurizer for rural supply

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LAWYER'S VIEW ON CONTRACTS Tunstead

INDUSTRIAL USE AT KINGSTON Helmrich

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1960 AWWA CONFERENCE
Bal Harbour, May 15–20

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Journal

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February 1960

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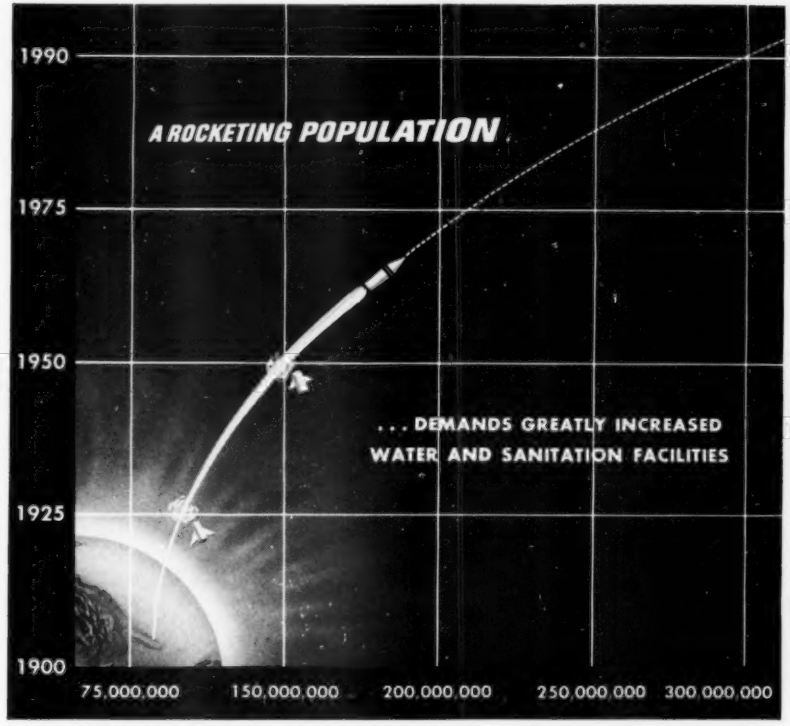

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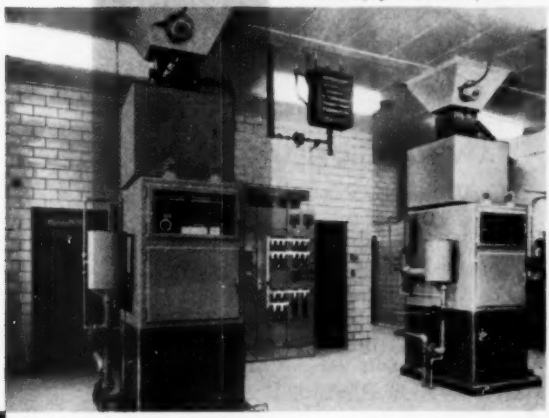

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AWWA ANNUAL CONFERENCE

Bal Harbour, Fla. May 15–20, 1960

See January issue of *Willing Water* for preliminary program.

Coming Meetings

AWWA SECTIONS

Winter-Spring 1960

Mar. 16–18—Illinois Section, at Pick-Congress Hotel, Chicago. Secretary, D. W. Johnson, Research Engr., Cast Iron Pipe Research Assn., 3440 Prudential Plaza, Chicago 1.

Mar. 17—New England Section, at Statler Hotel, Boston, Mass. Secretary, R. M. Soule, Assoc. San. Engr., State Dept. of Public Health, 511 State House, Boston, Mass.

Mar. 20–23—Southeastern Section, at De Soto Hotel, Savannah, Ga. Secretary, N. M. de Jarnette, Engr., Water Quality Div., State Dept. of Public Health, 309 State Office Bldg., Atlanta, Ga.

Mar. 24–25—West Virginia Section, at Hotel Pritchard (tentative), Huntington. Secretary, H. W. Hetzer, Engr., Union Carbide Chemical Co., Box 8361, South Charleston.

Apr. 5–7—New York Section, at New York Sheraton Inn, Binghamton. Secretary, Kimball Blanchard, New York Branch Sales Office, Neptune Meter Co., 2222 Jackson Ave., Long Island City.

Apr. 20–22—Nebraska Section, at Cornhusker Hotel, Lincoln. Secretary, J. J. Rossbach Jr., Civ. Engr., Metropolitan Utilities Dist., 3906 N. 48th St., Omaha.

Apr. 20–22—Kansas Section, at Broadview Hotel, Emporia. Secretary, H. W. Badley, Repr., Neptune Meter Co., 119 W. Cloud, Salina.

Apr. 22—California Section, at Mark Thomas Inn, Monterey. Secretary, R. E. Dodson Jr., Supt. of Water, Dept. of Utilities, Balboa Park, San Diego.

Apr. 24–27—Canadian Section, at Statler Hotel, Buffalo, N.Y. Secretary, A. E. Berry, 72 Grenville St., Toronto, Ont.

Apr. 28–30—Arizona Section, at Pioneer Hotel, Tucson. Secretary, A. D. Cox Jr., Secy. & Comptroller, Arizona Water Co., Box 5347, Phoenix.

(Continued on page 8)

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Coming Meetings

(Continued from page 6)

Apr. 28–30—Montana Section, at Northern Hotel, Billings. Secretary, A. W. Clarkson, Asst. Director, Div. of Environmental Sanitation, State Board of Health, Helena.

May 4–6—Pacific Northwest Section, at Benson Hotel, Portland, Ore. Secretary, F. D. Jones, W. 2108 Maxwell Ave., Spokane 11, Wash.

Jun. 14–17—Pennsylvania Section, at Hilton Hotel, Pittsburgh. Secretary, L. S. Morgan, 413 First National Bank Bldg., Greensburg.

Fall 1960

Sep. 12–14—Kentucky-Tennessee Sec., Knoxville, Tenn.

Sep. 14–16—Virginia Sec., Virginia Beach.

Sep. 14–16—New York Sec., Whiteface.

Sep. 21–23—Michigan Sec., Traverse City.

Sep. 21–23—North Central Sec., Sioux Falls, S.D.

Sep. 25–27—Missouri Sec., Jefferson City.

Sep. 28–30—Wisconsin Sec., Madison.

Oct. 9–12—Alabama-Mississippi Sec., Birmingham, Ala.

Oct. 16–19—Southwest Sec., Galveston, Tex.

Oct. 19–21—Iowa Sec., Des Moines.

Oct. 19–21—Chesapeake Sec., Washington, D.C.

Oct. 20–22—New Jersey Sec., Atlantic City.

Oct. 23–26—Rocky Mountain Sec., Colorado Springs, Colo.

Oct. 24–27—California Sec., Long Beach.

Oct. 26–28—Ohio Sec., Columbus.

Nov. 9–11—North Carolina Sec., Winston-Salem.

OTHER ORGANIZATIONS

Feb. 14–18—American Institute of Mining, Metallurgical & Petroleum Engineers, Hotel Statler, New York, N.Y.

Mar. 14–18—National Assn. of Corrosion Engineers Annual Convention, Memorial Auditorium, Dallas, Tex.

Mar. 16–18—23rd Short Course for Superintendents and Operators of Water and Sewerage Systems, Louisiana State University, Baton Rouge, La. Write: Fred H. Fenn, Dean, College of Engineering, at the university.

Apr. 3–8—6th Nuclear Congress, Coliseum, New York, N.Y., sponsored by EJC and other engineering and scientific societies, including AWWA.

Apr. 27–29—Conference on Algae and Metropolitan Wastes, R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: A. F. Bartsch, Asst. Chief for Special Technical Services, Water Supply & Water Pollution Research, 4676 Columbia Pkwy., Cincinnati 26, Ohio.

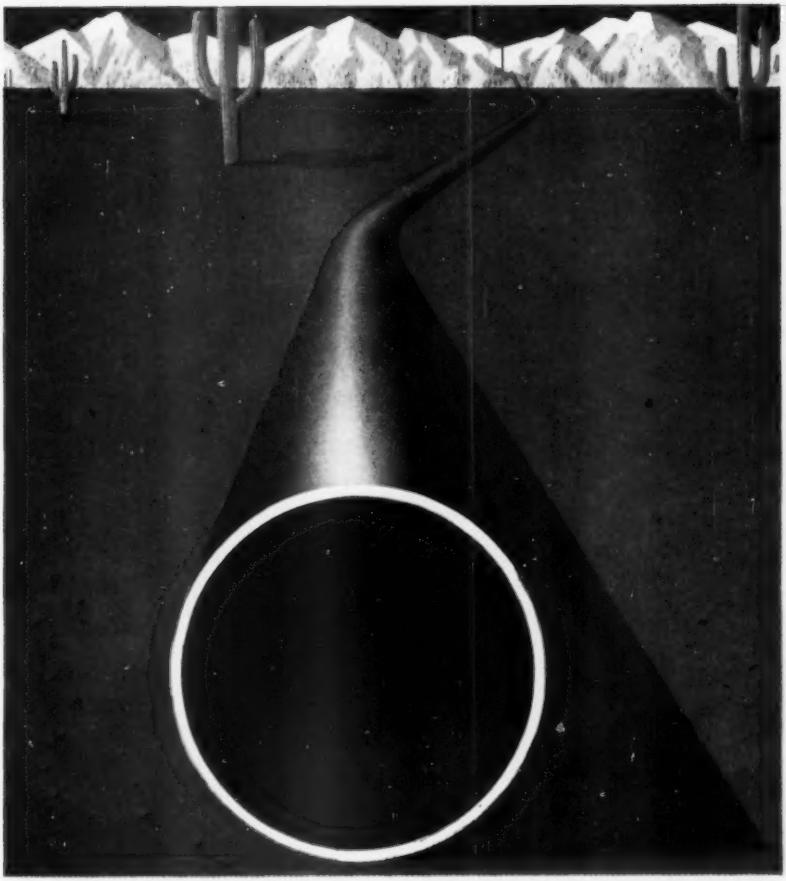
May 9–13—2nd Southwestern Metal Exposition & Congress, American Society for Metals, with other technical groups, State Fair Park & Sheraton Hotel, Dallas, Tex. Write: Allan Ray Putnam, Managing Director, Metals Park, Novelty, Ohio.

Jun. 1–3—5th Appalachian Underground Corrosion Short Course, West Virginia University, Morgantown, W.Va. Write: John H. Alm, Rm. 605, 2 Gateway Center, Pittsburgh 22, Pa.

Jun. 8–11—National Society of Professional Engineers, Statler Hotel, Boston, Mass.

Jun. 20–24—Summer general meeting, AIEE, Atlantic City, N.J.

Jun. 26–Jul. 1—ASTM, Chalfonte-Haddon Hall, Atlantic City, N.J.



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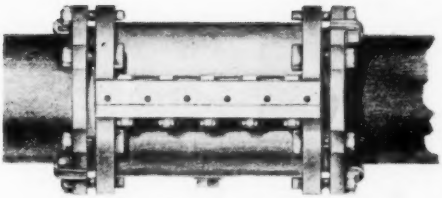
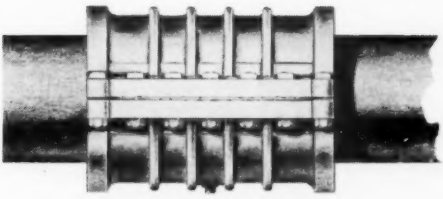


Mechanical Joint Repair Sleeve



Bell (Hub) End Repair Sleeve

INSTALL A SMITH SLEEVE

Smith Repair Sleeves are the answer to permanently repairing and quickly returning broken and cracked piping to service. Two types of Smith Repair Sleeves are available for installation on standard classes of cast iron pipe. Smith Mechanical Joint Repair Sleeves are produced in sizes 4″ thru 12″; Smith Bell (Hub) End in sizes 4″ thru 48″. Unskilled labor can speedily install Smith Mechanical Joint Repair Sleeves even in wet excavations. Bell End Repair Sleeves are most frequently installed with caulked lead joints. Smith Repair Sleeves reenforce the broken — cracked pipe and their service life equals the life of the pipe.

63

FIRE HYDRANTS · VALVES SINCE 1896

THE A.P. SMITH MFG. CO.

EAST ORANGE, NEW JERSEY

DON'T BE FOOLED BY INITIAL PIPE COSTS...

Certain factors concerning the type of pipe to be used for a proposed water or gas project must be examined carefully:

First — how much does the pipe cost, compared to other types?

Second — how often will it require repair?

Third — how long before it has to be replaced?

Sometimes the first cost of jobs where cast iron pipe is specified, is higher than similar projects using cheaper pipe. Yet, in the long run, cast iron pipe costs *less!* Here's why:

- Cast iron pipe rarely requires repairs. Its rugged construction, corrosion-resistant qualities and bottle-tight rubber-ring joints will withstand the most severe pressures.

- Cast iron pipe is built to *last*—98 American cities will testify to that. They've had cast iron pipe installations in constant use for over a century! Once cast iron pipe is in the ground, it stays there!

Don't be fooled by "low-cost" pipe. Insist on the pipe that will actually save you money over a period of years.

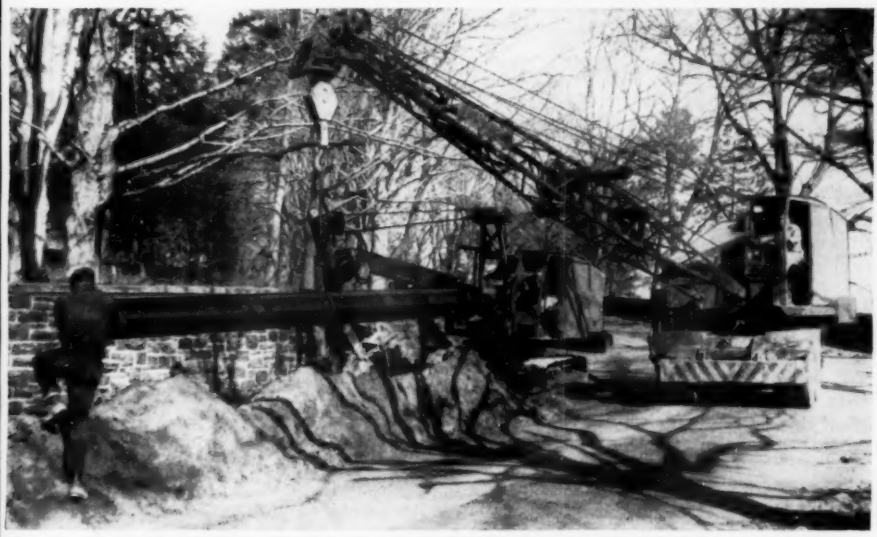
In Nebraska—Here a section of cast iron pipe is being relocated. Twenty-five years old, the pipe is still in excellent condition—has never required major repair . . . or replacement.

Rely on CAST IRON PIPE

In Indiana (above)—"All-weather" cast iron pipe is quickly installed despite wet trench conditions. Slip-on joints were easily assembled, with one workman using a crowbar.

In Kansas (above)—36" cast iron water main. Another part of this main was floated out of its trench by a heavy downpour. Later a 150 psi water test revealed no leaks in the slip-on joints!

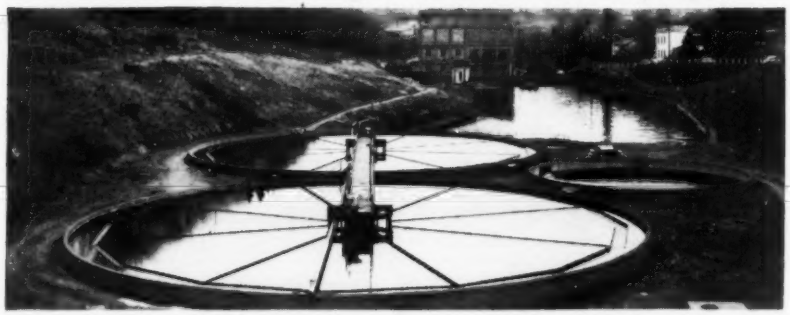
In Pennsylvania (below)—This 16" cast iron pipe is being installed as fast as the trench hoe can prepare the trench. Handy lengths and slip-on joints make cast iron pipe easy to handle, even in crowded neighborhood sectors; require less labor.



CAST IRON PIPE
THE MARK OF THE 100-YEAR PIPE
Cast Iron Pipe Research Association, Thos. F. Wolfe
Managing Director, 3440 Prudential Plaza, Chicago 1, Ill.

HAGAN COAGULANT AID PRODUCES STABLE SLUDGE BLANKETS, IMPROVED FLOC

at South Pittsburgh Water Company



Here's how South Pittsburgh Water Company overcomes the tremendous problems posed by making Monongahela River water fit for domestic use. The water contains industrial wastes, sewage pollution and acid mine drainage—hardness ranges from 50 to 400 ppm and pH fluctuates continuously.

Serving more than 350,000 people in an area of 200 square miles around Pittsburgh, the company uses two clarification-softening systems, a conventional sedimentation plant and two upflow units.

Hagan Coagulant Aid #18, used in both systems, has produced these four main results:

1) REDUCTION OF CARRYOVER—Coagulant Aid #18 is particularly effective in cold winter months during low softening periods. Its effectiveness is evidenced by continuously good floc formation, despite reduced feed of softening chemicals.

2) EFFECT ON SLUDGE AND FLOC—the aid produces a tougher, heavier floc, stabilizing sludge blankets. This improves water clarification and reduces problems caused by carryover.

3) IMPROVED PLANT EFFICIENCY—the rapid change of raw water quality makes hourly laboratory control mandatory. Hagan Coagulant Aid helps reduce clarification problems to a minimum—permitting plant chemists to concentrate on other vital laboratory procedures.

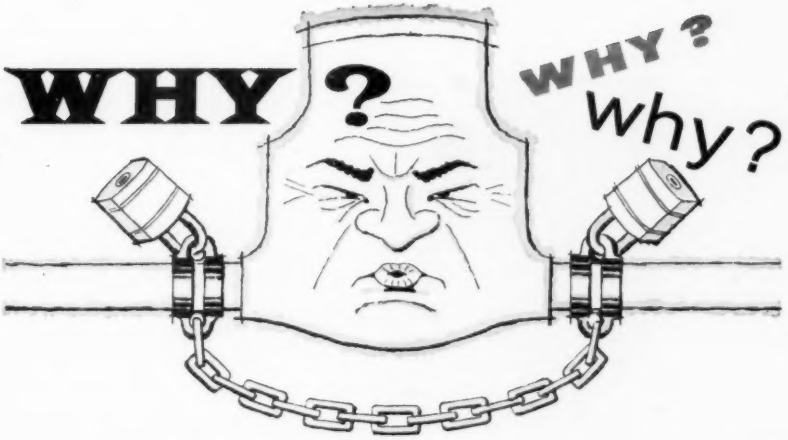
4) BONUS BENEFITS—by stabilizing sludge in the clarifiers, Coagulant Aid reduces total alkalinity which, in turn, leads to substantial savings in required softening chemicals.

Let Calgon Company tell you how their wide range of Coagulant Aids can play an important part in your water plant operation. Contact your nearest Calgon office or write:

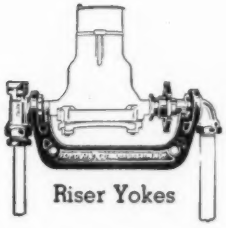
CALGON COMPANY

HAGAN BUILDING, PITTSBURGH 30, PA.

DIVISION OF **HAGAN** CHEMICALS & CONTROLS, INC.



SHACKLE YOUR WATER METERS?



Riser Yokes

Angle Yokes

Straight Line Yokes

Ordinary old-fashioned meter couplings are frequently like shackles on your water meters. They often make meter changing so difficult and troublesome that periodic testing is omitted and proper meter maintenance neglected.

Be sure to set your meters the modern way. See that Ford Yokes are keeping every meter available for fast, fast, FAST changing and testing. Savings in time and increases in revenues can be really substantial.

You can change a meter in a Ford Yoke

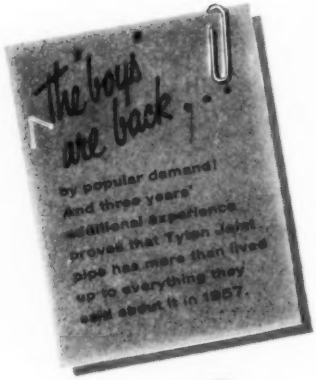
EASILY

There are models for every condition. Send for free catalog.

FORD

FOR BETTER WATER SERVICES

THE FORD METER BOX COMPANY, INC. Wabash, Indiana


The boys are back!
by popular demand!
And three years' additional experience proves that Tyton Joint pipe has more than lives up to everything they said about it in 1957.

a cinch to

Tyton Joint® pipe is *almost* as easy to install as our hillbilly friend indicates. Only one accessory needed . . . a specially designed rubber gasket that fits into the bell of the receiving pipe. A push or two and the connecting pipe compresses the gasket . . . seals the joint bottle-tight and permanently.

No bell holes. No waiting for weather. Tyton Joint pipe can be laid in rain or wet trench. It's so simple, in fact, even an inexperienced crew quickly becomes exper

U.S. PIPE FOR WATER, SEWERAGE ANI

nstall!

ou'll be hearing more about this ingenious new
yton Joint. Why not get the facts firsthand . . . and now?

rite or call. We'll be glad to give them to you.

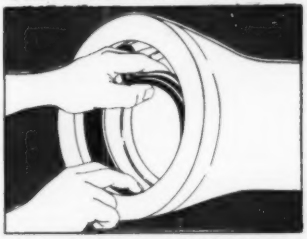
. S. PIPE AND FOUNDRY COMPANY
eneral Office: Birmingham 2, Alabama

WHOLLY INTEGRATED PRODUCER FROM MINES
ND BLAST FURNACES TO FINISHED PIPE

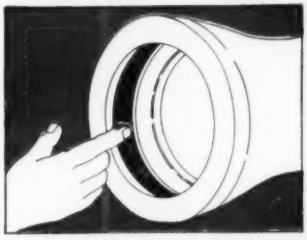
NDUSTRIAL SERVICE CAST IRON

TYTON

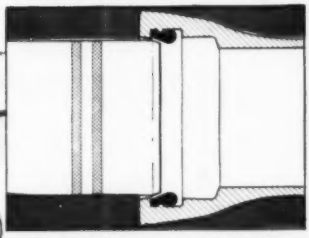
ONLY FOUR SIMPLE ACTIONS



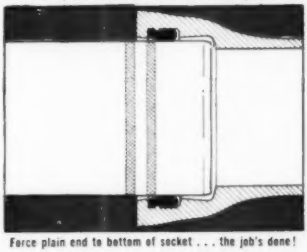
Insert gasket with groove over bead in gasket seat



Wipe a film of special lubricant over inside of gasket



Insert plain end of pipe until it contacts gasket



Force plain end to bottom of socket . . . the job's done!

what price water?

WHAT PRICE WATER? is a 12-page, 4x9-in. booklet that calls to the attention of the reader the real value of public water supply. Comparing 1940 prices of a number of other common items with today's, it also presents a comparison of the 1940 costs of water works facilities with present ones as an indication that water rates must be boosted. Designed to fit in a standard No. 10 business envelope, the booklet sells at prices ranging from 10¢ to 2½¢ per copy. Imprints of your name and address as well as your rate per thousand gallons or cubic feet can be provided on lots of 500 or more. Ask for sample.

Willing Water Jewelry

Since their introduction in 1954, the Willing Water jewelry items have been most popular as awards, gifts, and good will builders. Presenting a blue-faced Willie in full stride, the emblem has made a hit as a lapel button as well as a decoration on a number of jewelry items. Included in the list of items now available are:

AW-1	Lapel Emblem (screw-back)	$ .75
AW-2	Lapel Pin (joint pin & safety catch)	.75
AW-3	Key Chain (spiral mesh chain and spring-lock holder)	1.25
AW-5	Zippo Cigarette Lighter (brush finish, boxed)	3.15
AW-8	Shortie Tie Clip (rhodium plated alligator clip)	$1.85
AW-9	Money Clip (rhodium plated)	2.75
AW-10	Cuff Links (rhodium plated disks)	4.50
AW-11	Earrings (screw type)	2.50

Willing Water Service Buttons

The popularity of Willing Water lapel emblems has led to the design of a special button for recognition of tenure. The design shown at the left was prepared on the request of the Alliance, Ohio, Water Department, which now uses the service buttons. It is now available to you, with your company name engraved on it. Minimum order is 25 buttons or pins. On such an order a die charge of $17.50 is made for inserting your company name. The pins (with joint pin and safety catch) or buttons (with screw-back) are priced as follows:

Bronze	$1.00	10K Gold	$5.25
Sterling	1.50	14K Gold	6.75
Gold filled	2.00		

The minimum quantity (25) may be assorted as to years of service and as to metal used. Please specify clearly the number of each type required with pins (for female employees) and with screw backs.

AMERICAN WATER WORKS ASSOCIATION

2 Park Avenue **New York 16, N.Y.**

EXTRA SAFETY FACTOR!

CONSERVATIVE DESIGN is one of the important reasons why American Concrete Cylinder Pipe has gained such high acceptance by engineers and water works officials in the west. The composite design of the pipe is based on a factor of safety of 2¼ at the elastic limit of the pipe. American Concrete Cylinder Pipe will safely withstand sudden and severe increases of pressure – surge and water hammer – or the occasional concentrated external loadings met under unusual field conditions.

The *composite* action of the mortar lined steel cylinder, circumferential rod reinforcement and external mortar coating accounts for the consistently successful performance record of this type of pipe. The use of a minimum of 25% to, under certain conditions, 50% or more of the total required cross-sectional steel area as tension-wound circumferential reinforcement substantially increases the pipe's external supporting strength over that which would be obtained if all of the required steel area were placed in the cylinder alone.

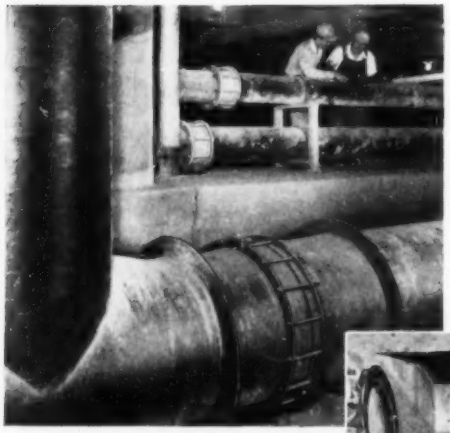
Take advantage of American Concrete Cylinder Pipe's greater inherent strength through more effective use of materials – see an American sales engineer when planning your next project.

CONCRETE PIPE FOR MAIN WATER SUPPLY AND TRANSMISSION LINES

American PIPE AND CONSTRUCTION CO.

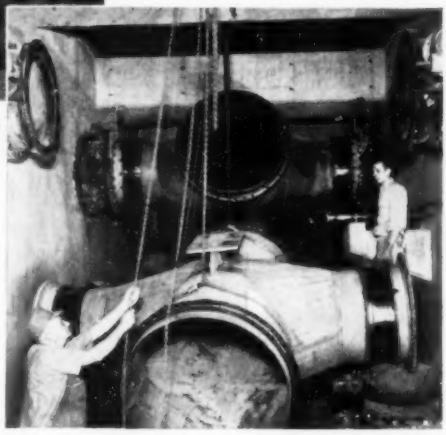
LOS ANGELES: 4635 Firestone Blvd., South Gate, Calif.— LOrain 4-2511 • **HAYWARD:** P.O. Box 630 — JEfferson 7-2072 • **SAN DIEGO:** P.O. Box 13 — CYpress 6-6166 • **PORTLAND:** 518 N.E. Columbia Blvd. — BUtler 5-2531 • **ALBUQUERQUE:** P.O. Box 1782 — CHapel 7-0486 • **PHOENIX:** American Concrete Pipe Co. (Subsidiary) P.O. Box 12127 — Alpine 2-7566

Joining pipe the Dresser way...



Always the easy way...

The 8-inch, 10-inch and 30-inch Dresser-Coupled lines illustrated here are part of the system used in servicing the Clague Road's advanced rapid-sand type filters.



often the only way!

This battery of steel crosses is in the new Clague Road Filtration Plant, designed by Havens and Emerson, Cleveland, Ohio. The Plant will carry a normal flow of 50-million gpd. Contractor: Hunkin-Conkey Construction Company.

Dresser® Couplings provide the essential leeway for bolting up cross flanges to valves in tight spots. They'll take deviation caused by settling concrete and remain bottle-tight... permanently! Dresser's vibration-proof properties protect valves and other expensive equipment, and protect the pipelines, too. With a Dresser Coupling, two man-minutes per bolt gives you a permanent, leak-proof joint that absorbs expansion-contraction movement, keeps maintenance problems to a minimum, and makes repairs easier if and when they do become necessary. For a free copy of "Dresser Couplings for Filtration Plant Piping" and the Dresser Sewage Treatment Plants booklet, write to Dresser Manufacturing Division, Bradford, Pennsylvania.

Bradford, Pennsylvania

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Houston
New York
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MANUFACTURING DIVISION

ONE OF THE DRESSER INDUSTRIES

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These ~~464~~ 495 plants* know . . . the list keeps growing!

How to get to the bottom of backwash problems

The best way is to install Leopold Glazed Tile Filter Bottoms, just as all these plants have done.

Like them, you'll find Leopold Bottoms successfully meet all underdrain requirements. The individual Leopold blocks can't corrode or absorb water . . . are acid and alkali resistant . . . are not subject to tuberculation . . . and provide equal filtration and uniform wash distribution. In addition, because Leopold Bottoms won't wear out and assure minimum media maintenance, they are the most economical in the long run.

We'll be glad to supply facts and figures without obligation.

**Including over 100 rehabilitation projects*

F. B. LEOPOLD CO., INC.
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Exclusive Canadian Representative: W. J. Westaway, Ltd., Hamilton, Ont.

Leopold

Tapping Concrete Pressure Pipe... Simple, Safe, and Efficient

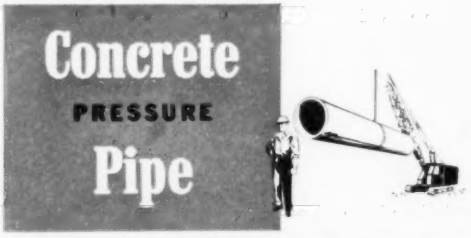
Modern equipment and techniques have made tapping of concrete pressure pipe under pressure a run-of-the-mill operation for the average pipeline maintenance crew. Pressure tapping of this type of pipe has become so commonplace that one private water works company alone has installed nearly 2,000 taps in its concrete pressure lines during the past 10 years.

Crews responsible for the operation find that tapping concrete pressure pipe under pressure is simple, efficient and, above all, safe—with no cross-threading or damage to the pipe.

In addition, concrete pressure pipe, itself, is so reliable that tapping is probably the only operation that a maintenance crew will have to perform on most concrete pressure pipelines throughout generations of trouble-free service.

For complete information, write to the American Concrete Pressure Pipe Association for its comprehensive booklet on the pressure tapping of concrete pressure pipe.

WATER FOR GENERATIONS TO COME



AMERICAN CONCRETE PRESSURE PIPE ASSOCIATION

228 North La Salle Street, Chicago 1, Illinois

From ⅝" service connections to 36" cross connections, pressure tapping by users of concrete pressure pipe is an easy and routine operation.

AMERICAN "AIRCOMB" DIFFUSER

WITH SELF-CLEANING NON-CLOG TEETH

Gives All the Aeration You Want . . . *just where and when you want it*

Teeth *on both sides* of the "Aircomb" provide simple, multiple, variable-flow orifices for controlled aeration.

No "*blurps*" at low air flows.

Unique *peaked-dome* design prevents accumulation of solids.

Velocities are self-cleansing.

Variable diffusion capacity as high as 5:1 if necessary.

Nothing to break or need replacement or change.

Fully proved in the field.

NOTHING CLINGS, FLINGS, OR SWINGS

—A TRUE TRIUMPH IN SIMPLICITY, ECONOMY, DURABILITY, AND EFFICIENCY

The AMERICAN "*Aircomb*" has very useful application in pre-aeration, channel diffusion, and activated sludge installations. It is especially preferred where relatively high tank content agitation is required and where other types of diffusers present operating difficulties due to clogging.

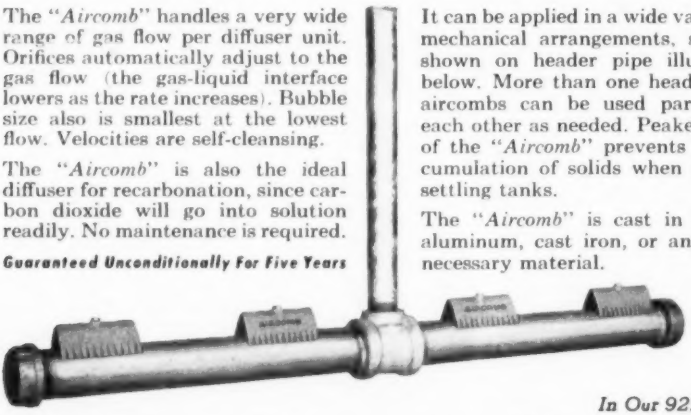
The "*Aircomb*" handles a very wide range of gas flow per diffuser unit. Orifices automatically adjust to the gas flow (the gas-liquid interface lowers as the rate increases). Bubble size also is smallest at the lowest flow. Velocities are self-cleansing.

The "*Aircomb*" is also the ideal diffuser for recarbonation, since carbon dioxide will go into solution readily. No maintenance is required.

Guaranteed Unconditionally For Five Years

It can be applied in a wide variety of mechanical arrangements, such as shown on header pipe illustrated below. More than one header with aircombs can be used parallel to each other as needed. Peaked-dome of the "*Aircomb*" prevents the accumulation of solids when used in settling tanks.

The "*Aircomb*" is cast in bronze, aluminum, cast iron, or any other necessary material.



In Our 92nd Year

Comprehensive Line of Field-Proved *PROCESSES AND EQUIPMENT*

The complete "American" line includes sewage, industrial waste, and water purification equipment, tank and pipeline mixers; deep well turbines; water transfer, clear liquid and waste water pumps. Consult "American" engineers or write for technical data.

AMERICAN WELL WORKS

112 N. BROADWAY — **AURORA, ILLINOIS**

Research, Engineering and Manufacturing — Waste and Water Treating, Mixing, and Pumping Equipment

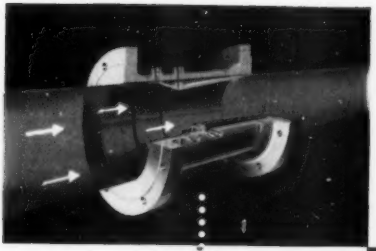
EST. 1868

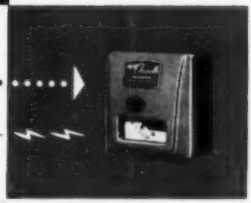
SALES OFFICES CHICAGO, NEW YORK, AND OTHER PRINCIPAL CITIES

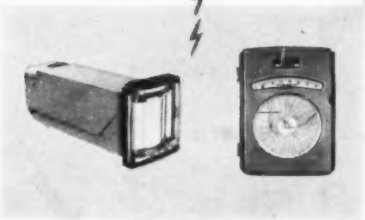
POSITIVE CONTROL OF MATERIALS IN MOTION B·I·F METERS FEEDERS CONTROLS

B-I-F COMPATIBLE COMPONENTS . . .

Increase efficiency and accuracy in INTEGRATED TELEMETERING SYSTEMS!







B·I·F *Builders-Providence* time-duration telemetering systems . . . primaries, transmitters and receivers specifically designed and teamed for better measurement and control of

- flows
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- temperatures
- levels
- positions
- weights
- electrical quantities

Primaries . . . most complete line available . . . Venturi Tubes for highest accuracy, Dall Flow Tubes for minimum head loss, and wide selection of special tubes and nozzles to suit any requirement.

Transmitters . . . many Chronoflo types . . . individually designed to transmit flow, level, pressure, electrical quantities, temperature or similar functions over long distances.

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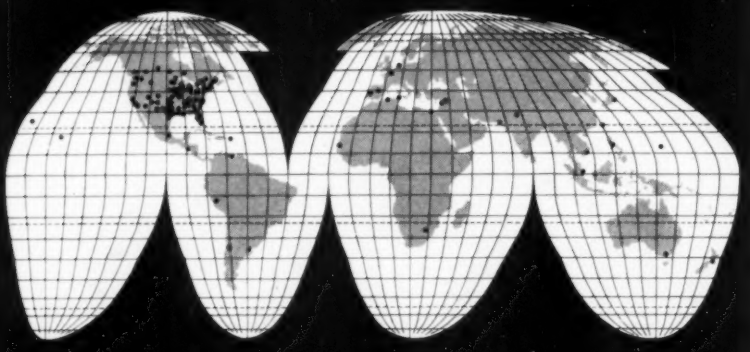

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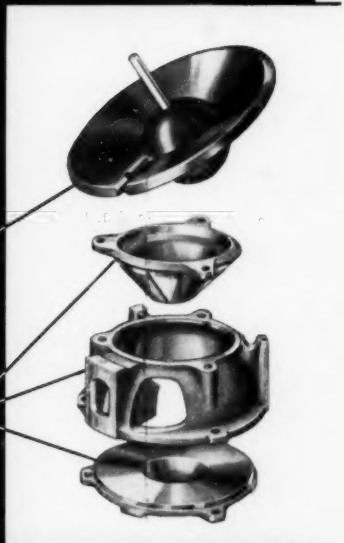

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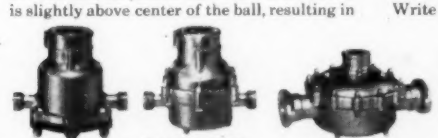

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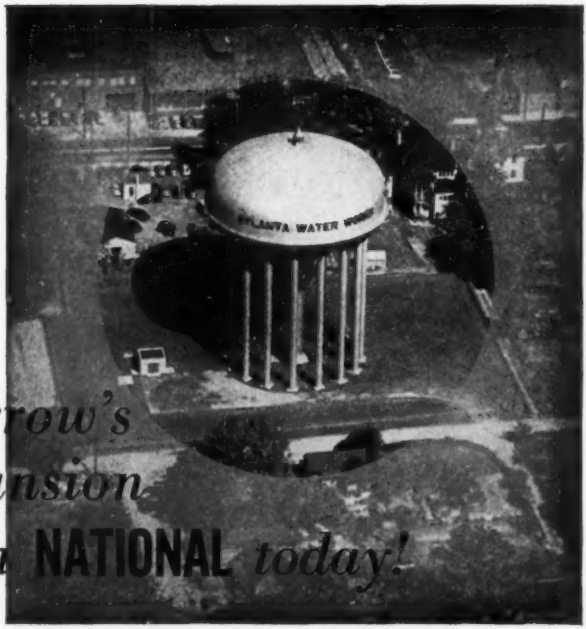

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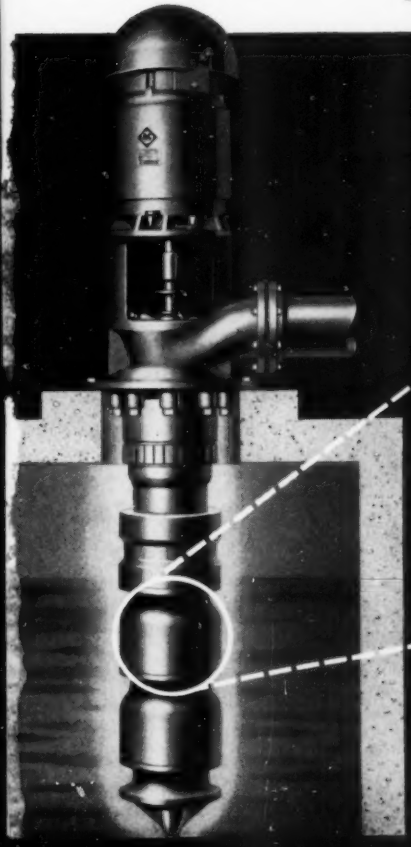

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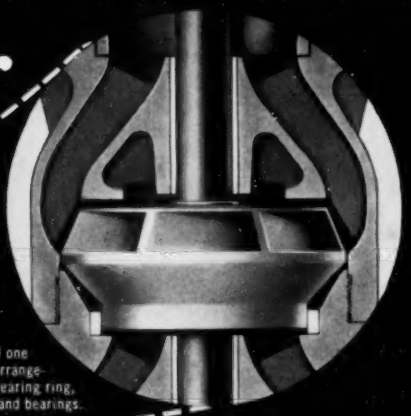

Cross section of one stage showing arrangement of bowl, wearing ring, impeller, shaft and bearings.

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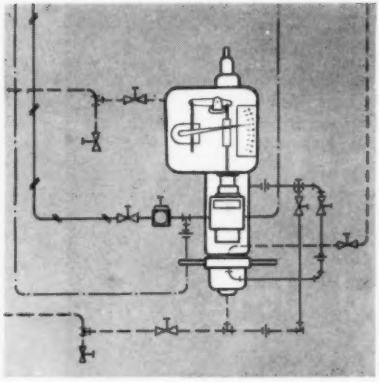

FLUIDICS AT WORK

New pneumatic transmission system more accurate, more dependable

The diagram shows a typical setup for the new Simplex system for transmitting rate of flow and loss of head to a receiver.

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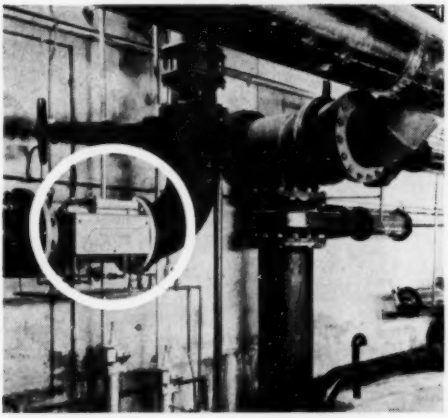

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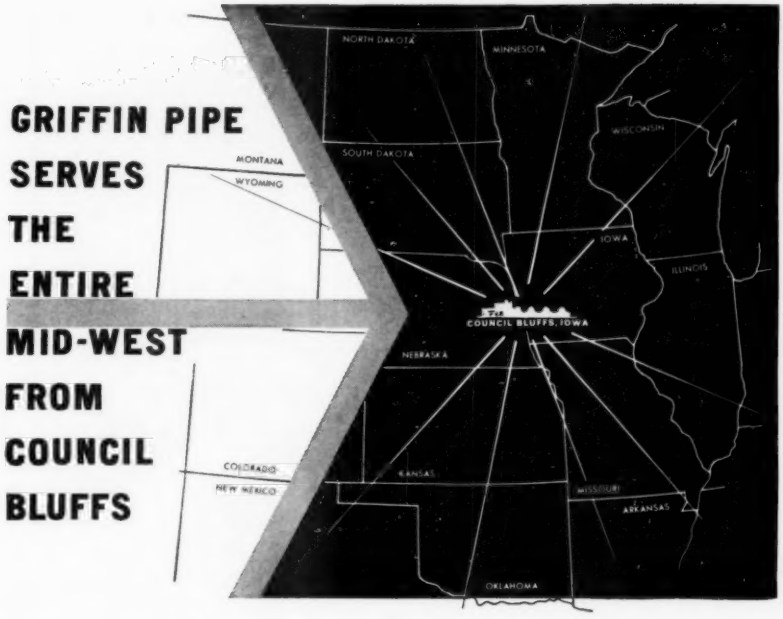

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AMERICAN WATER WORKS ASSOCIATION

VOL. 52 • FEBRUARY 1960 • NO. 2

Revenues, Rates, and Advance Planning

Louis R. Howson

A paper presented on Oct. 15, 1959, at the Iowa Section Meeting, Des Moines, Iowa, by Louis R. Howson, Partner, Alvord, Burdick & Howson, Chicago, Ill.

IT is self-evident that water revenues must be sufficient to cover the expenses of system operation and maintenance, accrue an adequate depreciation reserve to provide for necessary replacements and cover the interest and amortization requirements of outstanding debt. Up to this point, management ordinarily does a satisfactory job. No water utility is static, however, even in a community whose population is relatively constant; both water use and water quality standards increase as standards of living increase.

Importance of Advance Planning

A water utility can move only forward or backward; there is no standing still. Advance planning is absolutely essential to the provision of good service. It makes possible the most economical construction of the necessary facilities and is the best deterrent to the diversion of water utility funds into other channels. Advance planning reduces the necessity of financing by bond issues, which too frequently succeed or fail according to political influence, rather than on their own merits. Good planning also results in more consumer appreciation of service and in better public relations.

Growth and Needs

It has been demonstrated repeatedly that the managements of all water utilities should have present and future system requirements analyzed by someone qualified for the job (preferably a consulting engineer) at least as often as every 10 years. It is also generally agreed that, in normal times, any water utility that does not study and plan for its needs for at least 10 years in advance cannot expect to meet those needs when they materialize. These are not normal times, however.

In the past 10 years, as much money has been invested in water supply in the United States as in all preceding years. In fact, three-fourths of the total historical national water supply

investment, measured in dollars, has been made since 1943. Population growth is taking place at a rate of 3,000,000 per year, of which approximately 2,500,000 must be served by a public water supply. The provision of service to these new people alone would require an annual investment exceeding $600,000,000. Furthermore, present consumers are increasing their use of water by 200 mgd each year. The increase involves an additional annual expenditure of more than $250,000,000. In order to provide water service to 1,000 new consumers, a water system must have 2½ mi of new mains, 250 new service connections, 250 meters, and approximately 250,000 gpd of supply, treatment, and pumping capacity—not to mention additional storage and distribution facilities to meet peak demands. At 1959 prices, the investment required to provide service to 1,000 new consumers (250 metered customers) is approximately $250,000. Such requirements cannot be met successfully without adequate advance planning.

Financial Planning

Advance planning is more than simply studying the requirements of the future, outlining necessary construction, or estimating its cost. Because water utilities must finance their construction from revenues, whether directly or indirectly, it is just as essential to plan in advance for financing. It is even more advantageous to plan financing jointly with supply and construction requirements. Through careful coordination of plans for construction and improvements with plans for revenues and financing, better service can be provided at lower costs.

If considered far enough in advance, most ordinary construction can be so programed that it can be built from current earnings at a material saving in overall cost. For example, financing new construction with a 3 per cent bond issue of 20–40 years' maturity costs, when finally paid for, 35–75 per cent more than financing from earnings. Money is no longer available at 3 per cent, however. For revenue or general obligation bond financing undertaken at the present time (1959), the coupon rate is closer to 4½ per cent, and the total cost of the interest and amortization will be 1.5–2.25 times as much as the cost of direct financing from earnings. Of course, financing of capital additions from current revenues makes more liberal revenues necessary (about 25 per cent higher), but it offers a great net financial advantage.

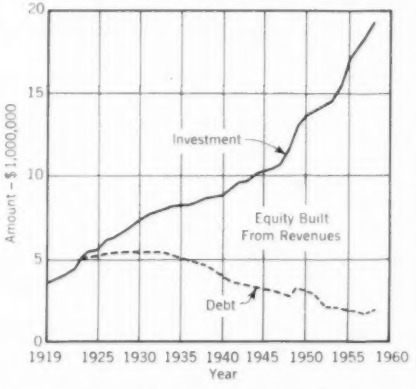

Fig. 1. Funded Debt and Investment of Des Moines Water Board

Since acquisition of the water utility by the city, the capital investment has increased approximately 450 per cent, whereas the funded debt, paid off entirely through revenues, has decreased to less than 40 per cent of its peak in 1930.

It should be clear, therefore, that it is economically desirable to have

enough surplus in the current revenues, after operation and maintenance costs, to pay for ordinary (and even extraordinary) construction. This can be done, if advance planning has made it possible to distribute construction costs rather than concentrate them in too short a time.

Planning at Des Moines

The Des Moines (Iowa) Water Board furnishes an excellent illustration of sound financing methods. The water utility was purchased by the city from a private company, in November 1919, for approximately $3,500,000. Bonds were issued to finance the purchase. At the present time the total indebtedness on the property is $1,833,000, but the book cost of the property as of Dec. 31, 1958, was $19,187,161. In other words, by coordinating its construction with revenues so that it could largely finance its construction without long-term borrowing, the city has been able, during 39 years of municipal ownership, to increase its equity in the water supply property from none in 1919 to 90 per cent in 1959. This was accomplished during a period in which the capital investment increased 450 per cent. The results of the Des Moines financing policy are shown diagrammatically in Fig. 1.

The Des Moines Water Board is now in the process of adding approximately $5,000,000 to its capital investment. In connection with this program, which will double the capacity of the softening plant, add a 75-mgd electric pumping station, provide several miles of feeder mains of up to 48-in. diameter, and build three distribution standpipes with an aggregate capacity of 8 mil gal, the Des Moines Water Board recommended and the city council approved an increase in water rates averaging 25 per cent. The construction program is to be spread over a 5-year period.

Instead of issuing long-term bonds, the board arranged for a short-term loan from a group of Des Moines banks in the amount of $1,800,000, at the exceedingly favorable interest rate of 2 per cent. Provision is made to retire all of the indebtedness from revenues by 1962.

The funded debt of the Des Moines Water Board, which is approximately 10 per cent of book cost, is only about 5 per cent of its present value.

Figure 2 shows the relation between invested capital and debt for the Louisville (Ky.) Water Co., another municipally owned utility that has done an outstanding job of advance planning and coordination of construction and financing programs. The Louisville debt is now about 17 per cent of the book cost and less than 10 per cent of its fair value, based on current construction costs and depreciation.

Capital Requirements

Of course, the amount of water utility revenue required for financing of capital expenditures at any one time depends on the state of the economy and the purchasing power of the dollar. In a study made by the author prior to World War II,[1] it was found from a study of the records of 100 water utilities that, for the period 1925–45, the investment in new construction averaged $1.34 per capita per year. During this period the *Engineering News-Record* (ENR) Construction Cost Index averaged 237 (1913 = 100). As of the summer of 1959 the ENR Index was about 800. The 1925–45 capital additions, in terms of current (1959) construction dollars,

would be approximately $4.25 per capita per year.

The growth in capital expenditures for water supply facilities during 1945–55, as analyzed by Harris F. Seidel and E. Robert Baumann,[2] shows that during that 10-year period the average growth in capital account had been $3.00 per capita per year for public plants and only $0.80 per capita per year for private plants, a weighted average of about $2.50. During the same period the average ENR Index was about 485. Thus, the construction of new water supply facilities did not keep pace with the general economy even during the 10 years following 5 years of war-restricted construction during which a large backlog of deferred construction accumulated.

The 1925–45 investment of $1.34 per capita per year was approximately 25 per cent of the gross water revenue during those 20 years. Des Moines, which had a book account of $10,238,966 at the end of 1945, added $8,948,195 in the next 13 years, the equivalent of $3.37 per capita per year. In those 13 years, the gross revenue varied between approximately $6.00 and $11.00 per capita, averaging $8.53. The expenditure was therefore equivalent to approximately 40 per cent of the revenues. During the same period, Des Moines financed its first 48-mgd softening plant from revenues.

At Louisville, where the capital account increased from $24,000,000 in 1946 to $46,000,000 at the end of 1958, the increase in capital account

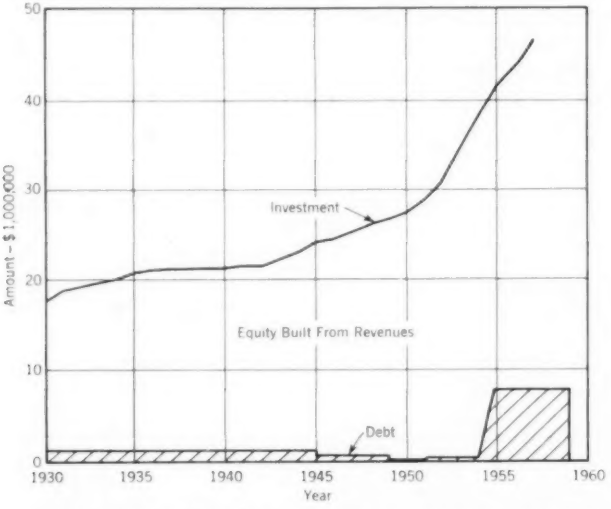

Fig. 2. Funded Debt and Investment of Louisville Water Co.

The growth in capital account during 1930–59 was $29,933,540; growth in debt was $6,921,000; equity growth (from revenues) was $23,012,540. Thus the percentage of capital account paid for through earnings during the same 28-year period was 77 per cent.

amounted to $3.91 per capita per year. Of the $22,000,000 capital additions Louisville built in the 12-year period, $14,000,000 was financed from revenues, which is equivalent to about $2.50 per capita per year, or about 30 per cent of the gross revenue. Thus, Louisville, with a book cost of $46,000,000, has only $8,000,000 of funded debt.

The financing of only normal, relatively continual construction, such as distribution mains, services, and meters, probably requires an average expenditure of $2.50 per capita per year at present (1959).

The total capital requirement of a growing water utility at the present time is approximately $1,000 for each new service. Probably about two-thirds of this goes for routine extension requirements and about one-third for periodic enlargement of supply, treatment facilities, and feeder mains. It is advantageous to finance both types of construction from earnings as much as practicable.

Rates and Regulation

Ordinarily, because the rates of privately owned utilities are controlled in most states by regulatory commissions, the public fails to realize that it is represented in the fixing of water rates. With the exception of a very few states, of which Iowa is one, the rates of all privately owned water utilities are fixed by public utility commissions. Only about one-fourth of all the water utilities in the United States are privately owned, however, and in only six states are the rates of municipal water works fixed or approved by regulatory commissions. Accordingly, the rates of 65–75 per cent of United States water utilities are fixed by the community they serve, usually by a city council. Members of city councils usually have very little familiarity with water supply in general, not to mention the financial requirements of water utilities. The responsibility for adequate water rates and financial stability in the majority of cities, therefore, rests primarily upon water utility managements, and it is their job to present the pertinent facts to those who set water rates. One of the most useful tools of management, in this connection, is a good annual report.

Some communities are fortunate in that the administration of the water utility is vested in a special board, usually with overlapping terms of offices to insure experienced leadership and comparative freedom from political interference. Most such boards do not have the authority to increase rates, but they are obliged to advise elected community officials of the financial needs of the water utility. When additional revenues are needed, it is the obligation of the board to be well informed of requirements so that they can present them convincingly to those who pass on rate changes.

Realistic Rate Structures

As rates for water service are (or should be) set well in advance of their effective date and are intended to provide for the future, it is the requirement of the period during which the rates will be operative that should be considered when rates structures are designed.

Operation and maintenance expenses are never static. Water utility operation and maintenance costs have increased from $2.40 per capita in 1925 to $3.35 in 1945 and $5.65 in 1955. The increase in per capita operation

and maintenance costs during 1945–55 was 69 per cent, or about 7 per cent per year. The increase in operation and maintenance costs has also been related to production. The Seidel-Baumann analysis showed operation and maintenance costs per unit production had increased 56 per cent in municipally owned water utilities and 94 per cent in privately owned utilities between 1945 and 1955. The weighted average increase was approximately 66 per cent, again nearly 7 per cent per year.

The effect of improved service upon operation and maintenance costs is illustrated by Des Moines and Louisville, where softening was introduced, and by Chicago, where filtration of part of the supply was begun after World War II. The data for these cities are shown in Fig. 3, together with comparable data for the *Engineering News-Record* Construction Cost Index. It should be noted from the graph that operation and maintenance costs per million gallons in these three plants that made changes in water treatment increased an average of 108 per cent for the 10 years, 1945–55, which is an average annual increase of 10.8 per cent per year. This increase is about 50 per cent greater than the average 1945–55 figures analyzed by Seidel and Baumann. Special factors, such as treatment improvements that may affect future operation and maintenance costs, vary greatly and must be considered in fixing future rates.

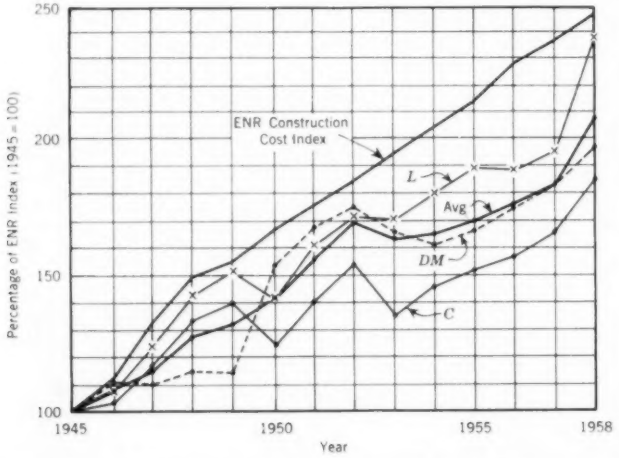

Fig. 3. Changes in Operation and Maintenance Costs for Three Midwestern Water Utilities, 1945–58

Cost comparisons are based on the cost per million gallons produced. Curves L, DM, *and* C *represent Louisville, Des Moines, and Chicago, respectively.*

Since the end of World War II, privately owned water utilities have suffered by the unwillingness of many regulatory bodies to recognize two things: (1) that dedication to public use begins when funds are secured and contracts are let for construction rather than when the facilities go into operation at some later date, and (2) that

operation and maintenance costs are likely to be materially higher when the facilities are finally put to use than when plans and hearings are begun. Rate hearings before commissions usually extend over a period of 6–12 months. As a result, the return allowed the company usually suffers substantial attrition before the new rates become effective; of course, the discrepancy increases as time goes on.

Some regulatory commissions still employ the original cost less the reserve as the rate base, even though, as shown in Fig. 4, the cost of reproducing older facilities is usually more than double the cost of the original facilities. It is, after all, the facilities rather than the dollars that are being used, and the dollars received for water service buy no more than other dollars; they must pay for improvements at present prices, not historical prices.

In view of all these things, it is not too surprising that during 1945–55, while municipally owned water utilities added $30 per capita in their investments, privately owned water utilities added but $8 per capita. This is further confirmation of the fact that adequate revenue is essential to, and a forerunner of, good service. During the same 10 years, the taxes paid by privately owned water utilities increased 115 per cent.

Failure to consider the fundamental factors outlined above, is to the public detriment. The public wants and is entitled to good service, but good service can be provided only under fair rates. One need only examine the comparative figures, which show that municipally owned water utilities have expanded nearly four times as fast as privately owned utilities, to see the results of backward policies on regulation.

Gross Revenue Requirements

In municipally owned utilities, estimates of the gross revenue requirement are based on:

1. Operation and maintenance costs (as estimated for 2–3 years in advance of a proposed rate change)
2. Debt service on obligations outstanding and to be issued in the succeeding 3–5 years

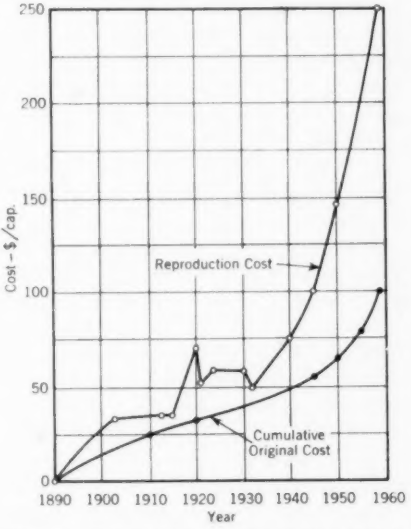

Fig. 4. Comparison of Cumulative Original per Capita Cost and Approximate Reproduction Cost of Water Works Construction, 1890–1960

Only a comparison such as the one above can be meaningful in analyses of water utility investment value.

3. A depreciation allowance (usually 1.5–2 per cent of book costs)
4. Estimated average annual capital additions for the next 5 years.

The sum of the four items gives the total revenue desirable, but it is not always entirely practicable to finance

the requirements from rates. The first decision must be how much of Item 4 to include in total revenue. When a decision has been reached, the total must be equitably apportioned between the various classifications of water users.

Rate Schedules

It is not the purpose of this discussion to detail how rate schedules should be developed; each situation has its individual characteristics. (The basic considerations are excellently set forth in the AWWA manual[3]). There are, however, a few criteria that apply generally:

1. A rate schedule with three or four user categories is preferred to one with several.

2. Where there is a "minimum bill" rather than a "service charge" type of rate base, the quantity of water in the minimum should be low, preferably 1,000–1,500 gal per month. In most systems, 40–50 per cent of all residential consumers use less than 3,000 gal per month. The Seidel and Baumann analysis showed that the average monthly minimum bill allowance was 3,200 gal for municipally owned utilities and 2,700 gal for privately owned utilities in 1955, both being about 10 per cent less than in 1950. The allowances are still so high, however, as to place a huge portion of the residential users on a flat-rate basis.

3. The cost of water in the highest bracket is usually 3–4 times as much as in the lowest bracket, and the two extremes usually are nearly equidistant from the average rate for all water sold.

4. Water can usually be sold to large customers at a much lower cost than to the average customer without inequity or unfair advantage to any group.

5. All rate studies require a classification of customer uses, and management can profit from gathering such data at frequent intervals. It is generally found that less than 1 per cent of the customers or less use as much as half of all water sold.

6. Every rate schedule should be as equitable as practicable, but some compromises must be made for purposes of expediency.

7. Rate changes need not, and usually do not, disturb customer relations, if the customers are advised, well in advance, of the necessity for the change and how it will improve service. The most readily acceptable change, if the existing rate schedule is already reasonably equitable, is an across-the-board percentage increase, applicable to all users.

8. The amount of the rate change is not of primary importance. Chicago has had two raises in 8 years of 50 per cent and 33 per cent, respectively, with no complaints.

9. It is the author's view that frequent rate changes are more annoying to the customer than a single large increase. Widely spaced increases of 20–50 per cent are to be preferred to more frequent increases of 10–20 per cent.

The customer inside the city should be supplied at cost. This includes proper provision for financing growth, of course, but fixed charges should be made only on outstanding debt. Users outside of the municipality should pay rates based upon a fair return on the fair value of the entire property. The application of such a utility rate base to users outside the city usually results in rates that are 50–75 per cent higher than city rates.

Summary

A public that appreciates good water service and is kept well informed of utility needs is usually willing to pay the cost of that service.

The actual difference in cost, to residential consumers, between mediocre and good water service rarely exceeds 0.5 cent per capita per day.

Every water utility, large and small, can afford and should have sound advance planning. Advance planning facilitates the financing of new construction from current revenues and results in a material saving in the cost of water service. Advance planning for construction and financing reduces the necessity for bond elections, which may cause consumer agitation.

Public water supplies must continue to grow. The money necessary to finance that growth must come from water revenue, whether directly or indirectly. Adequate revenue is, therefore, a prerequisite to good water service.

In general, city councils, which fix water rates for most United States cities, have authorized rates that permitted greater expansion than privately owned utilities, many of which are under unrealistic regulation, can afford. Publicly owned utilities have doubled their per capita debt in the last decade, whereas the privately owned utilities have kept their per capita debt almost constant. More than half the cost of plant additions in municipal water plants was by funded debt.

More municipally owned utilities would do well to finance construction with less borrowing, and it could be done, simply by more careful planning.

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Earnings of Municipally Owned Utilities in Wisconsin

Orville P. Deuel

A contribution to the Journal by Orville P. Deuel, Rate Analyst, Public Service Commission of Wisconsin, Madison, Wis.

WITH regard to the amount of earnings proper for a municipally owned utility serving citizens of the municipality—who might be compared with stockholders of a corporation—one might ask why there should be any earnings at all, inasmuch as the customers are also the citizens. One answer, of course, is that the citizens, as property owners and taxpayers, do not contribute to the community an amount commensurate with their use of utility service. It is conceivable that the citizen paying the largest tax may not be a customer of the municipal utility and that the best utility customer may not pay a significant amount of the taxes to the community. For these reasons, it is fair to have payments of utility rates cover the full cost of utility operation, and to have profits or dividends (appropriations to municipal funds) for the citizen owners. For convenience, appropriations to the municipality will be referred to as dividends.

An opposite view was expressed by S. T. Anderson,[1] formerly of the department of water, light, and power of Springfield, Ill.:

> A municipally owned utility should not be operated for profit, only to have this profit diverted to other departments of the city or village, but it should, through the water department, pay its share to the city. The fairest method of doing this is for the water department to pay to the city an amount equal to the taxes that would be paid by a privately owned utility, and for the city to pay for the service it gets from the water department. Money should not be taken from the water department and used by the city to reduce taxes.

A Wisconsin municipality that has decided to pay itself a dividend from utility earnings must make such payments in accordance with the provisions of the Wisconsin statutes. The use of funds of a municipally owned utility must be in accordance with Section 66.069(1) of the statutes. Income of a public utility owned by a municipality must first be used for operational, maintenance, depreciation, interest, and sinking fund requirements, local and school tax equivalents, additions, improvements, and other necessary disbursements, and indebtedness. Income in excess of these requirements may be used to purchase certain specified interest-bearing bonds or insurance on the life of an officer or manager of the utility, or may be paid into the general fund. Payments into the general fund are comparable to dividends paid by a corporation to its stockholders.

In addition to these provisions, the same section of the statutes contains a statement on the use that may be made

of water utility funds. This statement enumerates most of the items in the paragraph on all municipal utilities, but omits the statement on a tax equivalent. This omission may or may not show an intent to exempt water utilities from a tax equivalent. It is stated that water utility funds may be used for sewerage construction work, which is not mentioned in the section on utility income.

In order to discuss a payment that would correspond to the dividends of a privately owned utility, it is necessary to consider the entire relationship between the municipality as a governmental unit and the municipality in its capacity as a public utility. To do this, consideration must be given to charges for street lighting and other municipal uses of electric power, and charges for fire protection and other water service made available to the community. Also to be considered are the tax equivalent, depreciation expense, and the rate base of the utility, together with a return on the rate base.

Payments for Utility Services

It might be said that the payments for utility services rendered the municipality are commensurate with the benefit that the taxpayers derive from the utility tax equivalent and from a utility dividend paid into the general fund. This would be right if the amounts were equal, because it is true that the payment by taxpayers for utility service will increase taxes; the receipt of a water utility tax equivalent and dividend will reduce taxes. There are two important reasons why the problem cannot be handled in such a simple manner. First, a public utility, regardless of ownership, may not lawfully render free service; and, second, if such a policy were followed, the cost of providing the free service would fall on the ratepayers of the utility. Unless the amounts—that is, tax equivalent plus dividends—exactly equaled the value of free service provided, an unequal distribution of cost would result.

Tax Equivalent

The basis for a tax equivalent is found in the Wisconsin statutes. If there was at one time doubt as to the intent of the statutes with respect to a water utility tax equivalent, the public service commission has removed that doubt by establishing Sections PSC 109.01 to PSC 109.05 of the Wisconsin administrative code. The importance of the tax equivalent in municipal utility management is shown by the fact that tax equivalents charged to public utility operation by 166 Class A, B, and C municipally owned public uitilities in 1958 totaled $2,285,000. (In 1958, Class A and B utilities were those with revenues greater than $150,000. Class C utilities were those with revenues of $30,000–$150,000.)

Municipally owned property is exempted from taxation by Section 70.11(2) of the statutes; therefore, only the tax equivalent applies. Municipal utility plant and equipment—except for water plant—situated outside the municipality is taxed by the state department of taxation in the same manner as privately owned utility property located in more than one municipality.

The term "tax equivalent" raises the question "equivalent to what?". The public service commission has assumed that the term means an amount equivalent to the tax that would have been imposed on a public utility property not owned by a municipality. The understanding is that such property is as-

sessed on a full market value basis in the same manner as general property is assessed. The average of the tax rates of the various municipalities in the state is applied to the assessed value so determined.

The tax equivalent for municipal utilities could not be determined in exactly the same manner as for utilities assessed by the state. The method set forth in the state administrative code represents what is considered a workable compromise between the use of the average state rate and the assessed value and data available to local officials. The method used to find the maximum tax equivalent of a municipally owned utility relates the book value of the utility property to the general property assessment in the municipality by reducing or increasing the book value. This is done by multiplying the book value by the ratio of the local general assessment to the full value, as found by the state department of taxation. The actual local and school tax rates are applied to the assessed value so determined. With utility property taxed by the state, 65 per cent of the tax reverts to the local government. In most municipalities, local and school taxes exceed 65 per cent of the total tax rate. If the assessed valuation used for municipally owned utilities approximates that of a comparable public utility taxed by the state, the tax equivalent may exceed the amount that would be received by the local government from taxes on privately owned utility property.

Depreciation Expense

Another item to be considered in connection with any profits of a municipally owned utility to be paid into the general fund is depreciation expense. This is an annual charge made for the purpose of recapturing the investment in utility plant in equal annual amounts spread over the life of the plant. Funds so obtained may be placed in a separate fund and used to meet part of the cost of current additions and replacement of plant. Depreciation expense is important to a discussion of utility earnings, because the needs of the utility for funds to be used for capital improvements must be considered in connection with funds available for payment into the general fund. The fact that a utility has accumulated some cash and securities does not necessarily mean it has funds available for payment into the general fund. The accumulation or savings may represent accrued depreciation funds.

Return on Rate Base

A municipally owned utility is allowed revenues of an amount sufficient to provide a reasonable return on the rate base, in addition to operating expenses, a tax equivalent, and depreciation expense. The rate base on which the utility is permitted to earn a return approximates the book cost of utility property and plant, less the depreciation reserve and any customer contributions.

The basis for a return on the rate base of a municipally owned utility is stated in the statutes in Section 66.069 (1)(e): "Any city, village, or town owning a public utility shall be entitled to the same rate of return as permitted for privately owned utilities." For many years, the rate of return on the rate base of municipally owned water utilities has approximated 5.5 per cent; for electric utilities, 6 per cent.

In some instances, the return on the rate base has been confused with a return based on the capital paid in by the municipality account, sometimes re-

ferred to as the municipal equity. In terms of corporation vocabulary, the capital paid in by the municipality might be compared to common stock. As such, this capital would be a useful measure of the dividends paid to the municipality in terms of a return on the original cost of the capital stock.

Dividends

It seems that the municipally owned utility must, under the Wisconsin statutes, use its return on the rate base for additions and improvements when funds are needed for those purposes. The requirement would not prevent borrowing in any year in which there is a construction program of unusual magnitude, but borrowing by a municipality is governed by statutes and cannot be considered in the same manner as borrowing by a corporation. A privately owned utility may, and sometimes does, borrow money for construction or expansion, and the debt created will never be repaid. The bonds become a permanent part of the corporate financial structure, and interest is paid from utility earnings. Municipally owned utilities may issue only serial bonds for long-term obligations, and the bonds may not extend beyond a specified number of years. This means that after revenue bonds are issued, the municipally owned utility must, from its earnings, make regular payments on the principal and also pay the interest. This fact may explain the failure of many municipally owned utilities to make payment of a sum into the general fund in lieu of dividends that would be comparable with dividends of a privately owned utility. The payments to reduce the indebtedness will add to the surplus and, therefore, to the owner's equity, but such payments will use up funds that might otherwise have been available for dividends to the city or village. This may be a partial explanation of the fact that in 1958 only 12 of 87 Class A, B, and C municipally owned water utilities paid a cash dividend, and only 32 of 88 electric and joint electric and water utilities paid a cash dividend.

There should be no question as to the right of a municipally owned utility to pay a dividend if funds are available for that purpose. Funds will be available if rates are sufficient to provide a reasonable return on the rate base and if prior obligations for construction or debt service do not use up the earnings. It should be noted that as earnings are used for construction or debt retirement, the citizen stockholders will be enriched by the increased equity in the utility plant, which will be revealed in the surplus account.

Usually a municipally owned utility, as well as a privately owned utility, will prefer to maintain a relatively consistent dividend policy. Such a policy, in either utility, is within the discretion of management, with the limitations that there must be provision for funds needed to insure adequate service at all times and that a municipality must comply with the statutes on the use of funds.

Current Ratio

It is possible to use certain guides to determine when the balance between revenues and expenditures, including capital expenditures, is such that dividends may be paid without jeopardizing the financial assets of the utility. One such guide would be a device used by investors to judge the balance sheet of a corporation. The device, known as a "current ratio," is obtained by dividing current assets by current liabilities. For a public utility, cur-

rent assets usually consist of cash, government securities, accounts receivable, and materials and supplies on hand. The current liabilities include accounts payable, accrued taxes, accrued wages, accrued interest, and other accrued expenses. A desirable ratio at the end

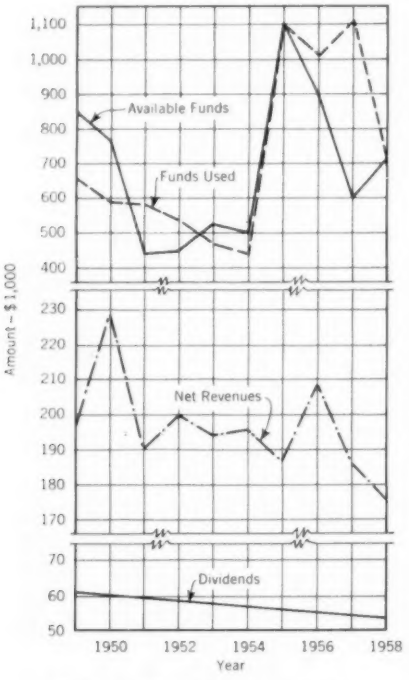

Fig. 1. Financial Records of Large Water Utility

Construction expenditures, included in the curve of funds used, varied from $290,000 in 1949, to $926,000 in 1956.

of the year for a relatively stable utility is comparatively low if bills are rendered monthly. A utility with a current ratio of three or better, and with no imminent construction program or needs, would probably be in a position to pay a dividend without jeopardizing utility operations. In any event, the utility's financial operations for several years should be surveyed to find the desirable ratio. If the current ratio falls below that found desirable, the operations should be examined to determine the cause. If the cause was temporary—an unusual expenditure not likely to recur, for example—or was the result of using cash for a completed construction program, the situation will often right itself. A corporation with an undesirable current ratio might continue to pay dividends for a time.

If the ratio is consistently and increasingly above the mark set as desirable, and no unusual construction expenditure is needed, the utility should either reduce its rates or increase the amount of dividends to its citizen stockholders. When the ratio falls below the desired standard and stays there, it is necessary to curtail the payments to the general fund or to increase rates for utility service.

General Fund

Whether the municipality pays available funds into the general fund as dividends on the investment in the utility, or deposits such funds in a reserve for future utility construction, or reduces its rates, is a matter of policy to be decided locally, as long as the return on the rate base is not considered excessive and funds are used in accordance with applicable statutes. If all available funds are paid to the general fund, it will be possible to lower taxes. On the other hand, if the funds are placed in a reserve for future utility construction, the possibility of better utility service is enhanced. If earnings above requirements are used to reduce rates, the municipality will be in a better position

competitively with other communities and, as a result, may attract industries and people. In this way, the municipality can increase its wealth and the base on which to collect taxes. It would seem that the question involved in a policy decision on the payment of dividends to the general fund by a municipally owned utility is not a simple one, and that at least the two alternatives mentioned previously must be considered by the management of the utility.

In 1958, only five Class A and B municipally owned water utilities paid dividends into the general fund; these dividends were relatively small. All 25 of these utilities—with gross revenues aggregating $17,000,000—paid tax equivalents of approximately $2,400,000, had earnings of $3,732,000, and paid $124,000 to the general fund in lieu of dividends.

A total of 36 Class A and B combined electric and water utilities, with gross revenues of about $18,000,000, had net revenues of about $2,500,000 and paid tax equivalents totaling $1,000,000. Of these, 16 utilities paid dividends to the general fund, which for the group totaled about $400,000.

A survey of 114 Class C utilities, including 52 electric or joint electric and water utilities and 62 water utilities, showed that only 23 of the 114 paid a dividend in 1958. Class C municipally owned utilities that paid money into the general fund included 12 of 43 joint electric and water utilities, 4 of 9 electric utilities, and 7 of 62 water utilities. Gross revenues of the 114 Class C utilities totaled $4,000,000; net revenues were about $1,300,000; tax equivalent, $750,000; and dividends, $92,000. A summary of these data is presented in Table 1, which shows that net operating revenues of 175 municipal utilities totaled $7,577,000, and the dividends paid were $619,000, or approximately 8 per cent of the net earnings.

TABLE 1

Revenue and Dividends of Municipally Owned Utilities—1958

Utility Class*	Utility Type	No. of Utilities	No. Paying Dividends	Net Revenues *$1,000*	Dividends *$1,000*
A and B	electric and water	36	16	2,518	403
A and B	water only	25	5	3,732	124
C	electric and water	52	16	649	48
C	water only	62	7	678	44
Total		175	44	7,577	619

* Class A and B utilities are those with revenues greater than $150,000; Class C utilities are those with revenues of $30,000–$150,000.

A second measure of the ability of a municipally owned utility to make payments to the general fund is obtained by a survey of present and future requirements, usually made by a competent engineer. The survey is the basis for estimating the utility's needs for capital funds. If the survey shows that needs can be met from earnings, any income in excess of requirements for construction would be available for dividends. If the needs for capital additions exceed the available funds, then borrowing will become necessary. If money is borrowed, then

the determination of the availability of funds for dividends becomes a matter of determining the utility's needs to meet debt service obligations. Any available amount in excess of requirements for debt service might be considered for payment into the general fund.

Financial Relationships

Records of several municipally owned utilities that usually pay a dividend were studied in order to determine their financial picture and the relationship between dividends paid and other demands on available funds. Some results are shown in Fig. 1–3. The curves for available funds include earnings, borrowings, and any increase in current assets. The curves for funds used include expenditures for construction, debt service, dividends, and a decrease in current assets. Differences from year to year can be ac-

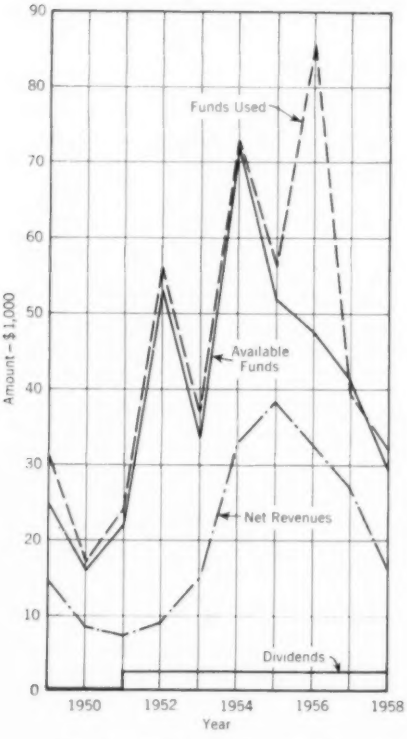

Fig. 2. Financial Records of Small Water Utility

The annual dividend represented 9–40 per cent of the net revenues.

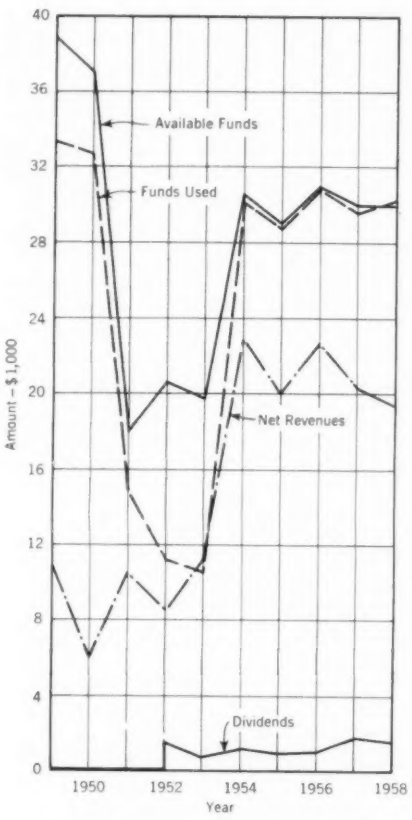

Fig. 3. Financial Records of Joint Electric and Water Utility

Net revenues plus depreciation expense exceeded construction cost plus debt service in each year that a dividend was paid.

counted for by the fact that current liabilities, such as accounts payable, may have shifted an obligation from one year to the next. Dividends of the utility were computed with a formula related to the municipal equity. Construction expenditures of the water utility shown in Fig. 1, which are included in the curve of funds used, varied from $290,000 in 1949, to $926,000 in 1956. Construction plus debt service of this water utility exceeded net income plus depreciation expense and contributions in 4 of 10 years in which a dividend was paid.

Figure 2 shows the record of a smaller water utility. The dividends were small and were maintained at a constant level. The annual dividend represented 9–40 per cent of the net revenues.

The utility whose records are shown in Fig. 3 is a joint electric and water utility. Net revenues plus depreciation expense exceeded construction cost plus debt service in each year that a dividend was paid. The amount of the payment to the general fund, referred to as a dividend, varied in an irregular manner; no information is available on how the amount was determined.

If a utility maintained a graphic representation of its financial picture, it could determine, with relative ease, the direction of its available fund balance and what must be done to maintain conditions under which a dividend may be paid to the citizen owners.

Summary

Dividends of municipally owned utilities must be paid in accordance with the statutes. The requirements for debt service will reduce the amount available to the general fund, as will new construction. A municipality owning a utility should determine a course to be followed and then balance the capital required for plant investment with funds available for all purposes. If a steady dividend policy is maintained, the utility will probably resort to borrowing in a relatively short time. The fact that some municipally owned utilities were acquired or constructed with borrowed funds and have little or no municipal equity, probably accounts for the fact that dividend payments are relatively few and do not constitute a high percentage of net revenues.

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Municipalities, Contractors, and Engineers—a Lawyer's Viewpoint

Richard H. Tunstead

A paper presented on Sep. 17, 1959, at the New York Section Meeting, Saranac Inn, N.Y., by Richard H. Tunstead, Partner, Spencer & Tunstead, Attorneys, New York, N.Y.

ALTHOUGH there is much formal literature on the laws concerning municipalities, contractors, and engineers, it is hoped that a lawyer's viewpoint on the relationships among these groups may be of value.

Municipalities

Municipalities are public corporations whose officers may be appointed or elected. In either event, they are limited in their actions by the laws that created their municipalities. The actions of officers are subject to political review and, in many instances, result in personal harassment. Legislatures have often attempted, with various statutes, to make municipal officers more honest than the average man. No matter how well intentioned these attempts are, they severely limit freedom of action. In addition, the officers generally are limited by the appropriations that have been provided for their activities.

It would seem logical, when anyone is dealing with a municipality, that he would inquire about the authority of the officer with whom he deals and about the availability of appropriations for the matter at hand, if payment is involved. Apparently, very few persons are willing to make such an inquiry; those who are not, must be prepared for tragic consequences. This precaution is pertinent to engineers in the early stages of planning a public work. It is suggested that, early in the planning phase, those dealing with municipalities make searching but discreet inquiries into the authority—financial and otherwise—of the various officials for whom the project is being planned. In dealing with municipalities, it is always wise to remember that moral commitments or informal or gentlemen's agreements with public officials are meaningless, not because these officials are immoral or ungentlemanly, but because they frequently may not have the legal power to make good on such commitments.

From time to time, public officials must supervise the construction of public works, which may be big projects. Few of these officials are selected on the basis of past experience with such tasks, yet the responsibility for the success of the project primarily rests on them. Obviously, this responsibility must be delegated to engineers and lawyers. Thus, one might also inquire about the qualifications of the people to whom responsibilities are delegated.

Many municipalities have salaried public engineers. In small municipalities these men prepare specifica-

tions for small public works, lay out roads and streets, plan curb and sidewalk work, plan road repairs, keep the local assessment rolls to date, work on the local master plan, and do the many odd jobs that local engineers can perform. It would be natural for a municipality that has such an engineer to turn its construction problems over to him. But unless the engineer is wise enough to know his limitations, this course could work out badly, because many projects require experience and know-how that the salaried, all-purpose engineer does not have.

Through the years, the engineering profession has become more and more specialized. There are highway engineers, sanitary engineers, water works engineers, foundation engineers, dam engineers, and many others. Public projects may require top engineering skills, which only a specialist has.

The consequences of using unqualified engineers in two particular instances known to the author may be of interest. In one instance, a municipality drew its water supply from a nearby river valley. Another municipality also wanted to use the same source. In such a situation, the right to take water from a valley must be determined by the state water power and control commission. The municipality that used the valley water asserted that there was not enough to supply both municipalities, and called as its witnesses two of the best hydraulic engineers in the country. For its witnesses, the other municipality called its local engineer and a local well driller to testify that there was ample water. The result of the hearing is obvious. The experts carried the day. Subsequent events showed that there was insufficient water for both municipalities and that the experts and the water power and control board were both correct.

In the second instance, the question before the court was the valuation of the Croton Dam. The nearby town was trying to uphold an assessment it had made on the dam; New York City, as owner, was trying to have the assessment value reduced and the accrued taxes refunded. During the trial, the town used as its expert the town engineer, who had never built a dam. He could not make his testimony stand up under cross-examination. The town lost. In this case, a local school district that would have had to pay a part of any tax refund owed to New York City had the decision reversed and a new trial ordered. On a retrial, using as its expert one of the foremost dam builders in America, the local authorities won their case. The amount of taxes involved in this litigation was more than $600,000. The expert's fee was about $3,500. This tells its own story.

Probably the failure of municipal officers to recognize the need for specialized engineering services is the basis of the often repeated expression, "small municipalities suffer from poor engineering." But the need for specialist engineers is beginning to be understood by public officials. Many small municipalities now have specialists doing their work.

Municipalities often expect too much from an engineer, who can only work with the tools he is given. Frequently, the municipality is unwilling to test materials properly or to pay for a special consultant. In spite of all, an engineer is expected to get his job done. Good public works are achieved with good engineering and adequate funds,

rather than with good intentions and insufficient budgets.

To avoid added costs where soil-boring is required, municipalities frequently state that the boring logs are not guaranteed, that they are not part of the contract documents. In such situations, a municipality calls on its attorney to draw up a contract under which the contractor assumes all risks of unexpected subsurface conditions. This does not work too well, because contractors have seen this trap too often and have learned to protect themselves from it. The books are full of cases when contractors have sued for damages for misrepresentation of subsurface conditions; and they frequently win, generally on the theory that the municipality had other information on the conditions which it did not disclose. The United States Army Corps of Engineers, probably the largest contracting unit in the country, has met the subsurface situation so many times that it has developed the well known "changed conditions" clause. It is a fact that this clause is one of the bases for low Army bids.

The author's law firm, after drawing up many public contracts, believes it good policy to make contracts so clear and so fair that the contractor knows exactly what he is supposed to do and bids accordingly. Experience has shown that this kind of contract has produced results far better than those from intricate contracts. As the saying goes, "There is no substitute for good engineering and fair contracts."

Contractors

A bid has been defined as a "darn good guess which a contractor will back with his money." A contractor studies the contract documents and estimates what he must do and what it will cost him to complete a job. To this estimate, he adds his overhead and profit; this is his bid. When he has doubts about an aspect of the job, or when he finds intricate clauses in the contract, he evaluates them as best he can, and resolves all doubts in his favor. The more risks imposed on him, the higher is his bid. To insure his profit, he generally overvalues the risks. If the risks that the bidder must take are reduced, the work will usually cost less.

A contractor also carefully considers the engineer in charge of the work. An unfair engineer is a risk that will cause the bid to be higher.

From time to time, a contractor entitled to extra compensation finds that he will not be paid for his extra work unless he sues the municipality. Some engineers will not admit error or incompleteness in their contract documents, and prefer to have a contractor sue the municipality and recover his money by way of a judgment. Perhaps municipal officers do this for political reasons. For instance, if they ask for and receive an appropriation of $5,000,000 for a particular job, and, because of unexpected conditions, the job costs $5,500,000, they can ask for a supplemental appropriation or let the contractor sue for his money. If they ask for an extra appropriation of $500,000, their cost planning may look bad. If the municipal attorney loses a suit to the contractor, that is the attorney's fault, not theirs.

Claims can be adjusted as the job progresses. It is less costly to settle claims before the attorneys get a hand in them and enlarge them to the allowable limit. Also, it is cheaper to deal with a friendly contractor than with

one who thinks that advantage is being taken of him.

The relationships between a contractor and his attorney, with regard to bidding and the fulfillment of construction contracts, merits discussion. Rarely, of course, does the contractor or his attorney draft the construction contract, although this sometimes happens in such specialized jobs as borings, pile driving, shoring, or underpinning. Some contract forms proposed to contractors or subcontractors contain unfair provisions that are shocking, but very often, in a private job, the contractor's attorney is able to rework the contract form and negotiate or "hammer out" an acceptable form. In public work, however, the contract is almost always drafted prior to the bidding, and all bidders must bid on the same contract. Usually, alert and successful contractors have their attorneys review the contract documents before the bidding to detect tricky clauses. Contractors also have their insurance advisers check the costs of all insurable risks and indemnification contained in the contract documents. Very often, because of conferences with these advisers, a request will be made to the owner's architect for an interpretation. Such an interpretation, freely and clearly rendered, often puts the bidder's mind at ease and generates closer bidding.

While the contract work is being performed, an attorney serves the contractor in a different way. When there are questions about the job, the contractor rushes to his lawyer, and the owner begins to receive "claim letters." These are often based on a well recognized principle that when an owner requires a contractor to do more work than is called for by the contract, or causes the contract work to be done in a more costly or onerous manner than called for by the contract, the owner is liable to the contractor for the extra cost incurred, plus a reasonable allowance for overhead and profit. Of course, rarely is the owner himself directly responsible for the extra cost; the engineer usually is, sometimes at the owner's direction or on his behalf. Contractors are aware of this extra-cost principle; so are their attorneys. But there have been instances when it seemed that the engineer or owner was unaware of it.

Engineers

One may wonder why engineers pay so little attention to their own contract rights. The author cannot recall being asked to draft many contracts for engineer clients, but has seen some very poor contracts engineers have made for themselves. It is also puzzling to find engineers and architects trying to draft public and private construction contracts. This is not an engineer's work, but an attorney's work; frankly, most engineers who try to draft a contract do not manage very well. The reason for this is simple. Engineers are disciplined to express themselves with formulas and figures; attorneys are disciplined in the use of words. In the drafting of contracts, clarity of expression is of primary importance.

A lawyer's advice to engineers might include:

1. Draw specifications so that they clearly tell the contractor what he must do, and under what conditions.

2. Ask the contractor to take no risks that good engineering methods can avoid.

3. As far as possible, work out payment schedules so that there can be

no unbalancing of bids. Contractors attempt to unbalance bids in order to get payment for their investment in the early stages of the work.

4. If there are errors or omissions in your plans, go to the owner, admit the error, and try to overcome it by a fair agreement with the contractor.

5. Require an attorney to take the responsibility for the preparation of the contract form itself and for seeing that it dovetails into your other documents.

6. Be fair and reasonable in your inspection and requirements.

7. Avoid the position, "I am only requiring the contractor to do what the book says he must do." This assertion generally means that you are requiring him to do far more than the contract requires. At the Wingdale Hospital near Pawling, N.Y., the plastering contract said that all corners must be square. The engineer said this meant square to 6 ft on each side of the corner. One never saw walls so square—but it cost the state hundreds of thousands of dollars. To require perfection where perfection serves no purpose is to waste money.

8. Treat the contractor fairly. If he knows before the bidding that he is dealing with a fair-minded engineer, the bid will be lower.

Attorneys

Many law firms specialize in work for municipalities, engineers, and contractors. One might inquire about the reasons that attorneys become interested in the construction field. Some young lawyers start their careers with, and remain with, firms that specialize in this field. Some attorneys study both engineering and law, and develop a following in the field. Others more or less drift into the field because some of their clients become interested in the construction industry.

Another inquiry might concern the qualifications that make a good lawyer in the construction field. Of course, he must be well trained in law. He must be a realistic person in order to understand the practical side of what is being done. He should be able to read and understand plans and specifications. If an attorney, in addition to representing contractors and engineers, is to represent municipalities, he must understand the problems of public finance and the laws applicable to the type of municipality or authority he represents. He must have some ability in the field of public relations, for attorneys, like engineers, must be able to inspire confidence.

Industrial Use and Conservation at Kingston

William Helmrich

A paper presented on Apr. 9, 1959, at the New York Section Meeting, Rochester, N.Y., by William Helmrich, International Business Machines Corporation, Kingston, N.Y.

THE author's firm, which is one of the large manufacturers of business machines and electronic equipment, has a plant located at 2 mi north of the city of Kingston, N.Y., in the town of Ulster. The city of Kingston has a population of 30,000 and enjoys a water supply of excellent quality, which it draws from a reservoir system through gravity feed at a rate of approximately 5 mgd. The pipelines of the Kingston system pass approximately ½ mi south of the plant property.

Prior to the construction of the plant at Kingston in 1954, it was decided and agreed upon by local and state officials that Kingston water could be made available in the quantities required by the plant. The city water lines were tapped with a single 12-in. feeder, which is the only water source for the plant; the plant is the largest single user on the city system.

The Kingston plant is located on a site of approximately 200 acres; the actual building area is 900,000 sq ft including manufacturing areas, offices, laboratories, warehouses, a boiler plant, an industrial-waste treatment plant, and some minor structures.

Water Uses

The major water requirements of the plant are (Table 1):

1. Sanitary use, cafeteria use, drinking water, vending machines, and other domestic uses

2. Industrial use, which is the largest use, in the plating area, the etched-card area, the paint shop, and the heat treat area

3. Boiler makeup water, cooling-tower makeup, and bearing cooling

4. Waste treatment, including all sanitation and industrial wastes (Wastes must be treated so that they can be safely discharged into Esopus Creek, which has a relatively small flow (20 cfs). The stream is used by farmers to water cattle, and it is used for fishing, swimming, and other recreation. The plant's responsibility for proper treatment is, therefore, very great. In the treatment of plant wastes, large quantities of dilution water are necessary to insure absolute safety of the discharge.)

TABLE 1

Breakdown of Water Use at Kingston Plant

Use Category	Avg Use *gpd*
General domestic and sanitary	80,000
Industrial	480,000
Boiler makeup and cooling tower	160,000
Waste treatment	30,000
Fire protection	
Roof flooding, lawn sprinkling, and miscellaneous	50,000
*Total**	800,000

* Varies from a summer maximum of 900,000 gpd to a winter minimum of 700,000 gpd.

5. Fire protection use, which, fortunately, is small (The potential use for fire protection has been provided for, however. Almost all of the plant is sprinkler protected and fire hydrants and hose cabinets are located throughout the plant site.)

6. Miscellaneous uses, such as roof flooding, refrigeration condenser cooling, and lawn sprinkling.

Plant Facilities

Figure 1 shows the layout of the plant and the location of the city water main entering the site. The buildings shown are the office buildings, the cafeteria, and the manufacturing buildings. The engineering laboratory on the other side of the street is a two-story building.

The water tank (Fig. 2) has a 200,000-gal storage capacity, half of which is reserved for fire fighting and cannot be drawn through the domestic loop. The piping hookup insures the maintenance of the fire reserve at all times. A heating coil in the tank protects against freezeups, and cathodic protection is provided to prevent corrosion.

Figure 3 shows the water distribution and fire loop for the plant. This is a better picture of the water distribution system in the plant. Under normal conditions, the feed comes from the city line and the tank merely floats on the system.

Conservation Practices

Because the plant has approximately 6,000 tons of air-conditioning capacity, immediate consideration was given to conserving refrigeration condenser cool-

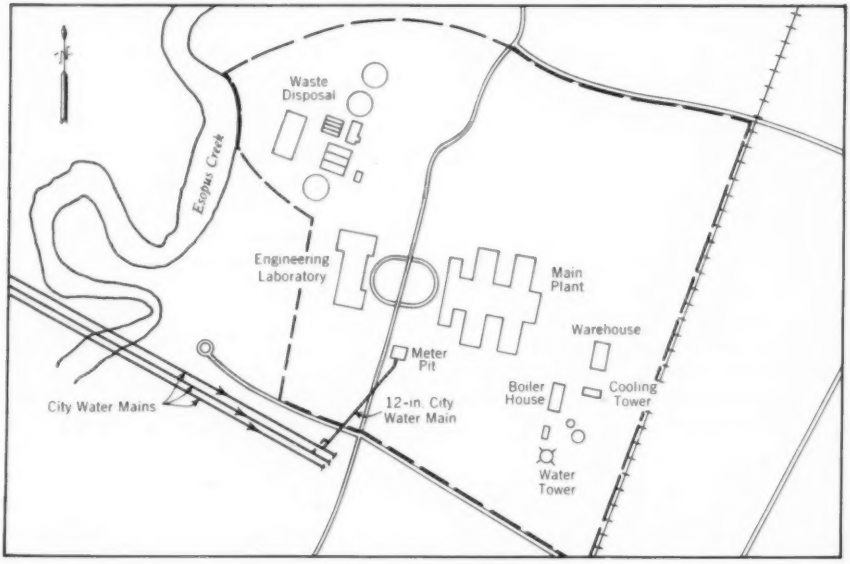

Fig. 1. Layout of Kingston Plant

The site outlined with the broken line is approximately 20 acres.

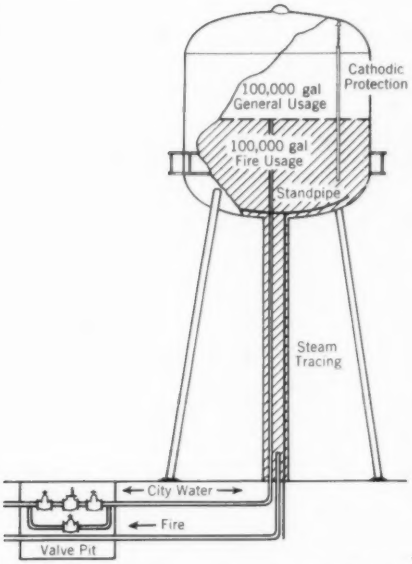

Fig. 2. Kingston Plant Storage Tank

Half of the tank's 200,000-gal storage capacity is reserved for fire-fighting and cannot be drawn through the domestic loop. The tank is heated and cathodically protected.

ing water. Without some means of conservation, more than 10,000 gpm would be required to satisfy the air-conditioning need alone. Approximately 20,000 gpm is recirculated and only 200–300 gpm must be used to make up for wind and drift losses, even during the hottest summer days. This is obviously a tremendous saving of water.

A stringent water conservation program has been instituted in other areas. It is good business to conserve water. When one considers that almost every gallon of water the plant draws from the city system must be handled again for proper treatment and disposal, it is apparent that a gallon saved in usage is also a gallon saved in treatment. Boiler water and cooling-tower makeup are the only large exceptions. Waste treatment costs approximately six times as much as the water itself. The plant has, therefore, instituted various water-saving devices in addition to the cooling towers.

1. In the plating and etched-card areas conductivity cell control is used. This device measures the conductivity of the water and permits only enough water to be added by a solenoid valve to keep the conductivity at a predetermined level. Without this device, water use in the plating area would probably be 1.4 times as high. In other words, the control results in a saving of 30 per cent in rinse water. Figure 4 shows a conductivity cell on a plating tank. The control is duplicated on 70 tanks in the plating and etched-card areas. The initial installation of the devices cost approximately $200 each. The 2-in. air gap

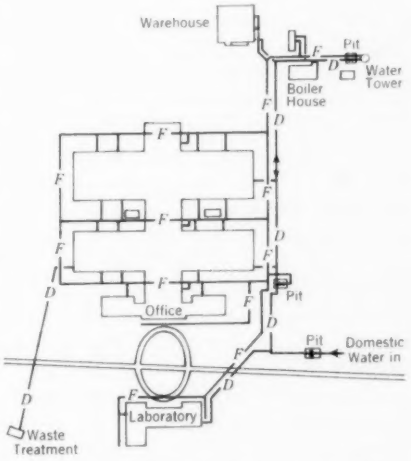

Fig. 3. Distribution Piping and Fire Loop

Lines labeled F *represent fire piping and those labeled* D *are domestic piping.*

should be noted, as it will be discussed later.

2. Conservation is also practical in the boiler house. Most of the water that is used for bearing cooling is piped to the cooling tower and used for cooling tower makeup. Thus, the water is used twice, resulting in a saving of about 35,000 gpd.

3. The air compressor no longer uses city water. Instead some of the chilled-water return for intercoolers and after-cooler cooling is used. The The saving is about 18,000 gpd. (City water was used in the beginning for cooling-tower makeup.)

4. Roof flooding is discontinued during periods of shortage. During the summer this saves an average of 48,000 gpd although there is some increased air-conditioning tonnage.

Figure 5 shows the use of a cascade system for plating rinse tanks to conserve water. The cleanest water is introduced in the final rinse and is allowed to cascade toward the first "dirty" rinse.

Safeguards

With the approximately 5,000 employees in the Kingston plant, and because the plant is directly connected to the city water system, it is obvious that a tremendous responsibility rests on the plant to maintain the quality of its system. The plant works very closely with the water department and the county health officer in order to insure protection. In the first place, the plant engineering department controls all changes or hookups in the water system. Safeguards such as the air gap provided on the plating tanks prevent the introduction of water directly into a contaminated solution. Similar safeguards are provided on every installation. Secondly, there are no cross connections between the domestic system and any other system, except that the fire loop comes off the

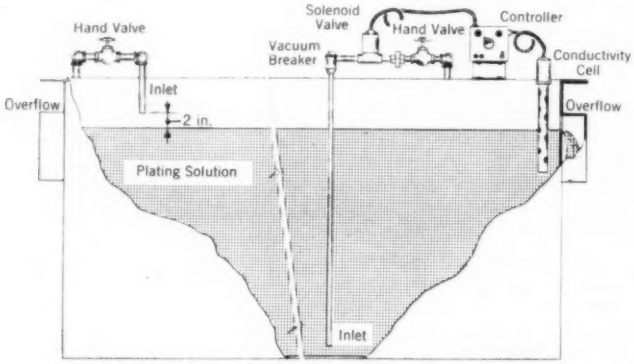

Fig. 4. Plating Tank With Conductivity Cell

The conductivity cell and control permit only enough water to be added to keep the conductivity at a predetermined level. This device effects a 30 per cent saving in plating water.

supply main through a double check valve. There are no cross connections with the chilled-water system, nor with the town supply passing the plant. Engineering personnel are constantly made aware of the need to safeguard the water supply.

The plant officials are aware that an unusually large draw on the city system might result in severe hardships for the city. Frequent discussions are held with officials of the Kingston Water Department to avoid such problems. During emergencies or periods of drought, special precautions are taken and constant contact is maintained with city officials.

Future Developments

The author's firm is constantly considering new ways to improve its efficiency in the use of water. Some of the measures now under study are stated below.

1. Some plating solutions may be reclaimed through the use of ion exchange. For example, contaminated chromic acid could be reused. The ion-exchange system would reduce water requirements for treatment.

2. Consideration is being given to changing plating procedures entirely and using different chemicals. Such a change might well result in reduced water treatment requirements.

3. A ground level storage tank of approximately 500,000-gal capacity (subject to revision) is proposed. Such a tank would provide increased fire protection and would be used as a reserve. It could also be used to provide service for several hours if the main supply were interrupted or reduced for any reason. Some pumps would obviously be necessary in addition to a tank.

Conclusion

It is self-evident that there is no one solution to the problem of how to make the most efficient use of water supplies in industry. The Poughkeepsie, N.Y., plant of the author's firm, for example, has two separate water supplies, one to satisfy the industrial need and another for domestic uses. Supply requirements vary with the type of industry and the location. The application of new techniques, and, most of all, increasing interest in the protection of the valuable resource, water, may soon make possible even more advanced methods of industrial use and conservation.

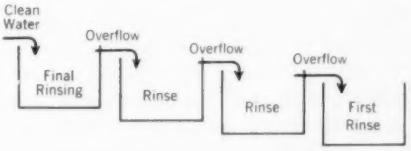

Fig. 5. Cascade Rinse System

The same rinse water is used four times, as shown, which effects a significant saving in plating rinse water.

Architectural Treatment for Water Utility Structures

Ronald Harrison and Paul M. Emery

A paper presented on May 6, 1959, at the Canadian Section Meeting, Montreal, Que., by Ronald Harrison, Mgr. & Secy.-Treas., Scarborough Public Utilities Com., Toronto, Ont., and Paul M. Emery, Chief Water Works Engr., Municipality of Metropolitan Toronto, Toronto, Ont.

THE subject of architecture in water utility installations is a very broad one, but there are three aspects of the subject that are of particular importance to water utility men: economic factors, land use requirements, and aesthetic considerations. Being a matter of taste as well as function, architecture is a very controversial matter, even among architects, and it is often possible to find as many opponents as proponents for any particular design. Therefore, in discussing aesthetic considerations, the authors can express only their own opinions (which, because there are two authors, were at times in opposition and required compromise), and there may be some disagreement with the views set forth in what follows.

Provisions for Expansion

Most water utility structures built in growing communities today can be expected, because of the nature of the services they perform, to have a useful life of 60–70 years before their replacement or abandonment is considered. For this reason, such structures should be designed for reliability and permanence, and in an architectural style which is pleasing today but not so radical that it will go out of favor quickly and be an eyesore for many years of its life.

Water utility men must plan for the expansion of the water system to keep pace with the growth of the community it serves. The availability of an adequate site within the required area, the choice of the height and location of buildings that will make them appear to their best advantage, the placement of structures on the site—in the light of such factors as possible street widenings (Fig. 1)—all are important considerations in design. The possibility of future expansion should be considered in the architectural planning of water utility structures. For example, a sectional design can allow the initial structure to appear complete in itself, and as additional sections are constructed, the buildings will continue to give the appearance of a complete plant (Fig. 2).

Economic Factors

The basic architectural factor governing the cost of water utility structures, particularly treatment plants and pumping stations, is the choice of materials. Intelligent arrangements of well chosen materials can result in a lower

overall cost during the life of the installation. By overall cost is meant the first cost of installation plus the cost of maintenance for a period of years. In other words, first cost should not be the only consideration. It is not unusual, in fact, when architectural materials or arrangements are chosen to provide ease of maintenance, for the first cost to be somewhat higher than if other materials or arrangements are used. In the opinion of the authors, however, it is economically sound to increase the initial investment in order to obtain those materials and architectural features which allow a considerable reduction in the expense of both labor and materials for maintenance. Some of the better materials and treatments used in water utility structures are discussed below.

Interiors

Interior walls of treatment plants and pumping stations can be effectively designed in a material such as glazed tile (Fig. 3), which requires a minimum of labor for cleaning and an almost negligible amount of repair. Even small installations designed within strictly limited finances may take advantage of concrete block or brick masonry, which is glazed on the interior face. Untreated interior masonry walls may last for some time without major care, but eventually they require painting. Once painted, repainting is usually necessary every 3–4 years with annual touching-up between times.

For interiors of filter plants, as in all very damp locations, plaster work on walls and ceilings should be avoided, as the cost of maintenance and repair may be unreasonably high over a period of years.

Floors may be of quarry tile (Fig. 4), terrazzo, hardened concrete, or a flexible tile of some kind, such as vinyl, depending on the location in the plant. Concrete should be treated so that painting is unnecessary, providing an easily washed and waxed surface.

In designing relief features on a flat masonry wall, whether interior or exterior, the architect should make his breaks vertical wherever possible. Horizontal checks in masonry provide ledges, which catch dirt and eventually contribute to discoloration of the surface.

Grilles, railings, and spandrels, whether inside or out, in corrosion-resistant metals, such as stainless steel or aluminum which require no painting, can also help to reduce the cost of upkeep for buildings (Fig. 5).

Windows and Glass

The size and location of windows are important factors. Unfortunately, quite often little thought is given to such matters beyond the overall aesthetic appearance in an architect's proposal sketch of the exterior views of the plant. Aside from first cost, the installation of a large number of unnecessary windows could lead to considerable expense for glass maintenance and replacement. If vandalism is prevalent in the area, the windows must be protected by screening, which, in addition to its cost, has a detrimental effect on the outside appearance of the plant and often destroys the effect the architect hoped for. Furthermore, the installation of a large glass area greatly increases the annual cost of heating for a building located in colder regions.

Effective use of structural glass block can give a pleasing architectural appearance and adequate lighting for

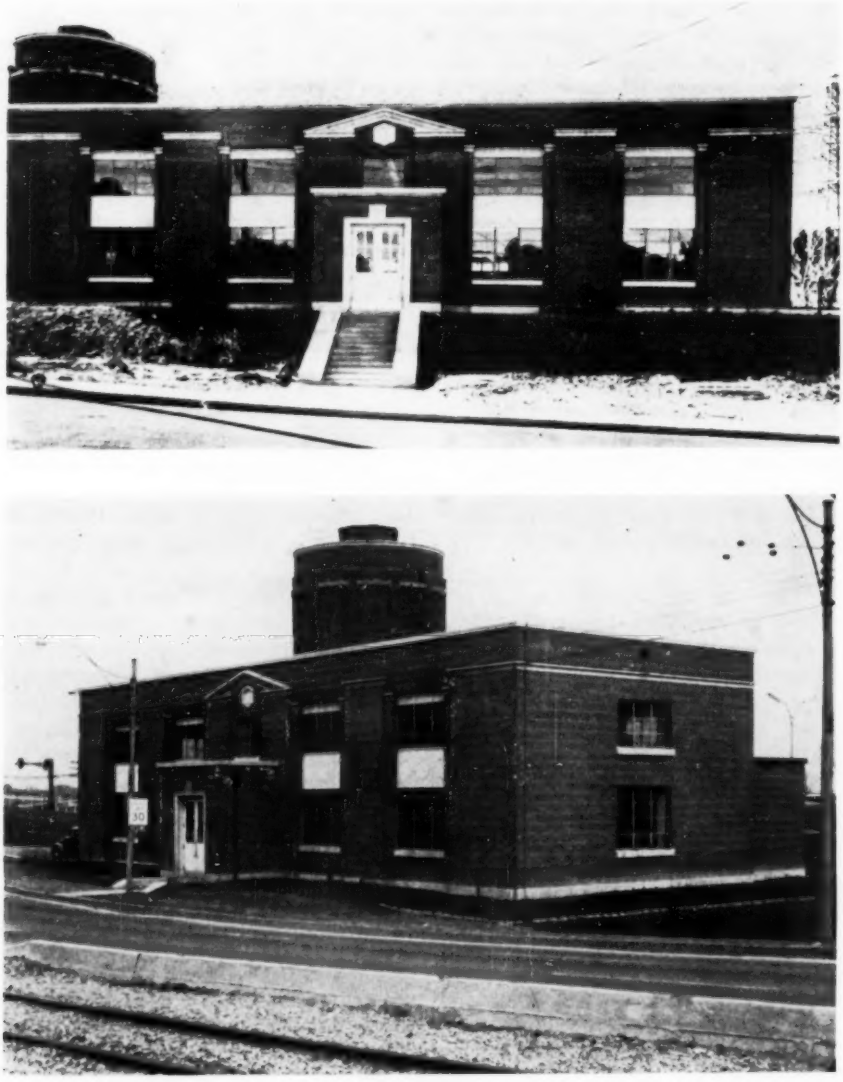

Fig. 1. Result of Improper Building Location

Failure to plan ahead for street widening pushed this pumping station within a few feet of the road. Sandwiched, as it is, between the highway and the railroad track, there was no way that this problem could have been avoided, except by choosing a larger site in the first place.

Fig. 2. Filtration Plant and Extensions, Scarborough, Ont.

The newer, extension portion of the building can be seen at the far left. This kind of sectional building makes expansion simple and economical, in addition to preserving the effect of uniformity in architectural style.

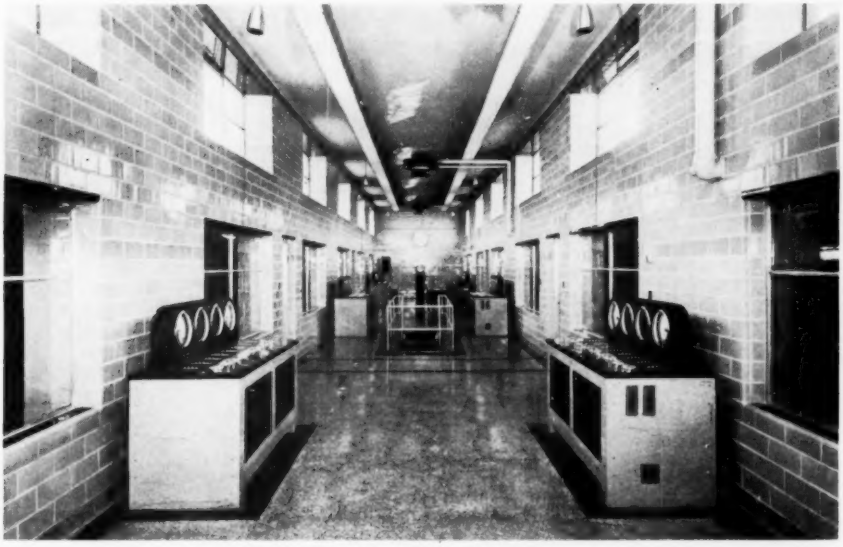

Fig. 3. Interior of Toronto Filtration Plant

Glazed wall tile and aluminum window frames reduce the cost of cleaning and maintenance. The placement of windows along the top of the walls also reduces the cost of daytime lighting.

the needs of a treatment plant (Fig. 6). Glass block requires little or no replacement and much less cleaning than clear glass windows. Heat loss is also reduced when structural glass block is used.

Land Use Requirements

A new water utility installation in a built-up or planned area must be designed with consideration for land use requirements and zoning regulations. Quite often a public utility—and, in particular, a water utility—because of the necessity for location in a given area, is permitted to contravene zoning regulations, provided that the architecture of the installation conforms, as far as is possible, with surrounding structures.

In industrial areas, where deterioration is continual owing to the proximity of industrial plants, consideration should be given to the use of materials which will give the longest maintenance-free service. For example, it would be pointless to construct a building of light-colored masonry adjacent to a steam railway switchyard, where discoloration due to smoke would cause the appearance of the building to deteriorate within a short period of time.

Present or proposed land use in the area also has some effect on architectural style. In a factory district, a treatment plant or pumping station may reasonably give the appearance of a factory building. Such a design will be less expensive than other choices,

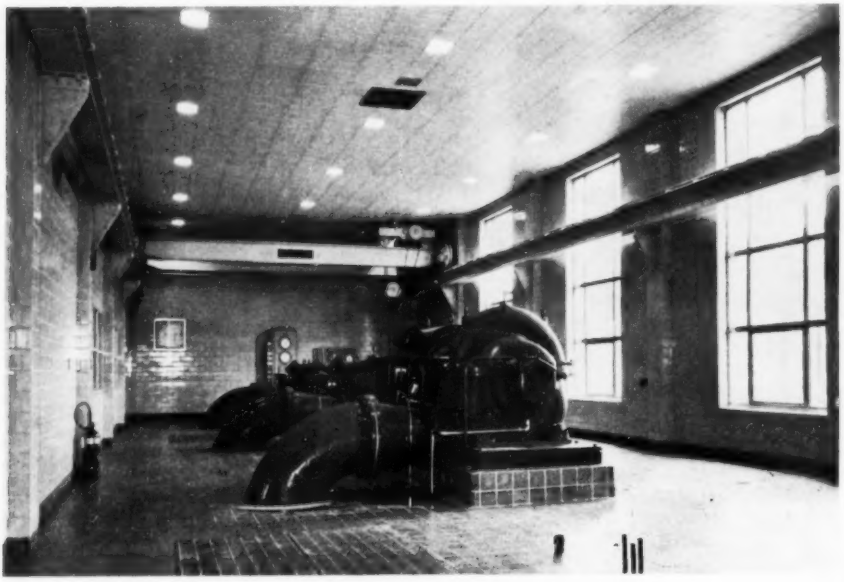

Fig. 4. Interior of West Toronto Pumping Station

Quarry tile is used here to its best advantage. The traveling overhead crane is an ingenious feature, in that it can operate over every part of the room.

and yet it will fit in with surrounding structures. In such a district, a beautiful, white stone building would be decidedly out of place.

In a commercial district, a water utility station should be a utilitarian structure—and one that is obviously a public building. Properly designed, such a structure could resemble a bank or a similar institution.

If it is necessary to construct a pumping station or a similar structure in a residential district, a definite attempt should be made to make it conform in type to the surrounding architecture (Fig. 7).

Office buildings, of course, vary as much in design as do residential homes. Water utility offices may be freely designed within the broad limits of the prevailing architectural standards.

Large water treatment and pumping plants, because they require large areas of land, may achieve a park-like set-

Fig. 5. Staircase at Ottawa, Ont., Plant

Proof that water utility buildings need not be architecturally uninteresting inside is the attractive staircase shown.

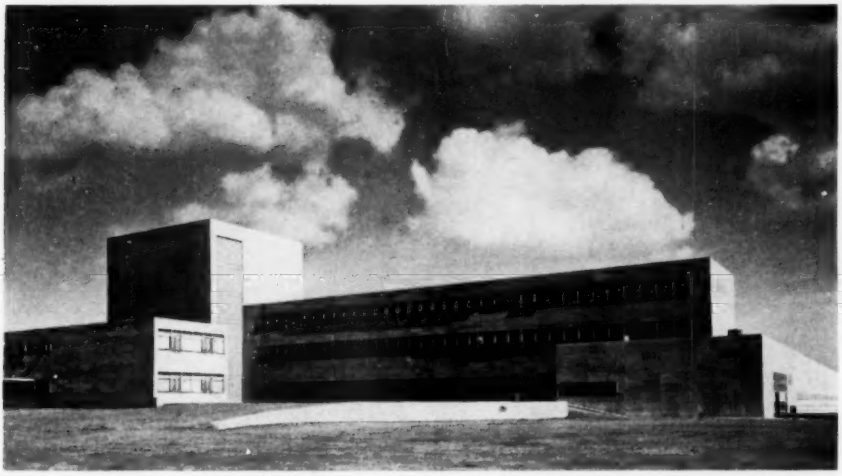

Fig. 6. Chicago South District Filtration Plant and Administration Building

One of the most impressive of all treatment plant structures, this building is a fine example of efficient and aesthetically pleasing design. The use of structural glass brick on the lower wall of the filtration wing (right) provides a maximum of light and reduces heat loss.

ting which makes architectural conformity with the surrounding area somewhat less important. Such installations, therefore, lend themselves to more individuality in design, within the limits imposed by the functions to be performed in the plant and the physical relationships that exist between such functions.

Aesthetic Considerations

From an aesthetic standpoint, the authors feel that there has been some tendency in the past to build water utility structures that are ugly and box-like, being little more than a weather-proof covering for the equipment (Fig. 8).

Whether the installation is made in an industrial, commercial, or residential area, some attempt should be made, by attractive design and the proper use of landscaping, to make it an object of community pride (Fig. 9 and 10).[1] Although this may involve some additional expense initially, a

Fig. 7. Front and Rear View of Toronto Pumping Station

Although it is usually a mistake to disguise the function of a water utility structure, zoning regulations in residential and commercial areas sometimes make it necessary. The building shown above is both functional and unobtrusive in a residential area.

neat, well-kept installation has a type of beauty which tends to increase the general tone of the neighborhood in which it is located, so that, within limitations, the owners of neighboring buildings will try not to be outdone to any great extent. An ugly building will, in all likelihood, be followed by similar structures, with the end result that the whole neighborhood will begin to deteriorate in a much shorter time.

If there is doubt as to the future character of the district in which a water plant is to be located, a simple, substantial structure with plain lines and a pleasing architectural form will be the best choice.

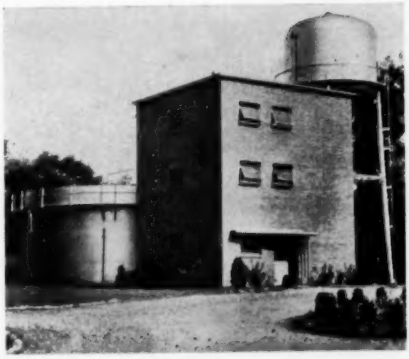

Fig. 8. Example of Unimaginative Building Design

This box-like building, although perfectly functional, is anything but attractive.

Fig. 9. Partly Underground Pumping Station

Built for a maximum of economy or semicommercial and lower-cost residential areas, this pumping station is unobtrusive and, because it is mostly underground, reduces machinery noise. Unfortunately, the necessity for storm fence and barbed wire detracts from its appearance.

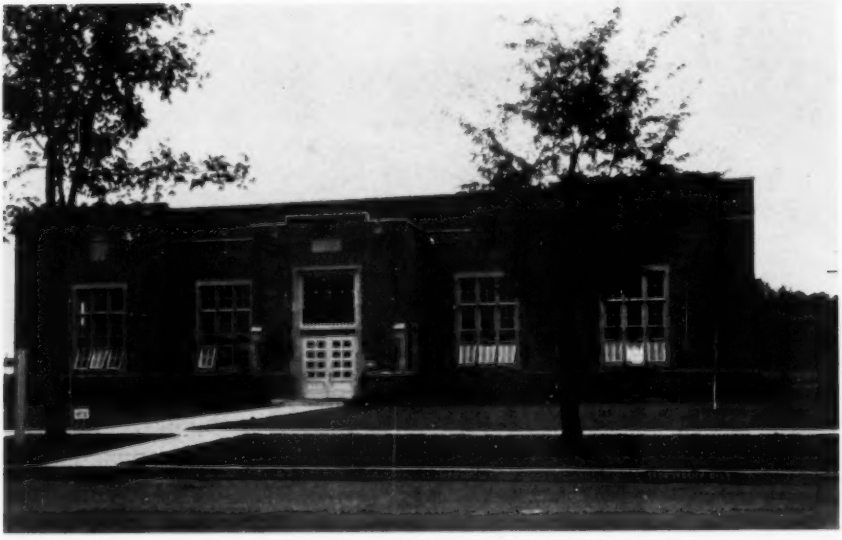

Fig. 10. Examples of Neat, Well Kept Installations

Although entirely different in style, both of the buildings shown above give the appearance of public buildings, both are neatly kept, and both are objects of pride in the community.

Elevated Tanks

Because elevated tanks are a special kind of water utility structure, discussion of them has been saved to the last. Tanks are essential to modern water supply systems, and, as steel does not possess the aesthetic versatility of masonry, the appearance of elevated tanks has often been subject to criticism.

Until a few years ago, water tanks were designed solely to hold a certain quantity of water at a definite elevation; aside from choice of color, no attempt was made to improve their appearance. The usual method of construction, which necessitated cross-bracing and guy wires, provided a functional but relatively ugly structure.

Recently, attempts have been made to achieve architectural beauty in the design of elevated tanks without disguising in any way their functional aspect. This has been achieved by the elimination of cross-braced girders and guy wires, by improvements in tank shapes, and, in more recent designs, by the elimination of all support columns other than the central pylon (Fig. 11).

Tanks are sometimes distinguished by special painting, as, for example, the single-pylon tank at Tam-O-Shanter Golf Course in Chicago, which is painted to look like a golf ball placed on a tee. Occasionally, elevated tanks have been so combined with other functions that the water storage use is completely disguised; for example, a 2.5-mil gal tank in Europe was recently constructed on a central pylon with the water tank enclosed by a restaurant at the upper level completely around the periphery.[2]

Fig. 11. Water Tank of Novel Design

The use of a single, central pylon for support is a fairly new development in elevated-tank design. This tank is the result of an attempt to improve the appearance of a steel tank without disguising its function.

Conclusion

The importance of good architectural design for water utility structures

should be obvious. Good architectural design, in the public utility field, means functional and economical design, of course, but it is particularly important that a building that belongs to the community present a pleasing enough appearance that its owners, the general public, can point to it with a feeling of pride.

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Plumbing and Public Health

In the June 1959 issue of *Sanitalk*, a publication of the Massachusetts Department of Public Health, an editorial discusses the close relationship between plumbing and public health, with special reference to research being conducted at the Plumbing Research Laboratory at the Lawrence (Mass.) Experiment Station.

Many instances of disease and fatality caused by back siphonage and cross connections introducing toxic substances into potable-water lines can be traced to improper plumbing installations. Cross connections have been a major cause of epidemic disease, and occasional flareups of intestinal disturbance, for which no other cause can be found, may well be traceable to cross connections with an unsafe supply. For example, cross connections of private wells with public supplies are common during periods of shortage. Underground watering systems introduce the possibility of back siphoning of weed killers, insecticides, fertilizers, or other toxic substances. The increasing number of private swimming pools also creates new hazards.

Copper poisoning has, in a few instances, been traced to the use of copper tubing in vending machines for carbonated beverages. Failure of check valves permitted carbon dioxide to back up, attacking the copper tubing. The first drinks drawn then contained toxic amounts of copper.

Improper plumbing is largely due to inadequate training of those engaged in plumbing design and installation, inadequate plumbing codes, lack of enforcement of the codes, or the installation of plumbing by unlicensed persons. It is hoped that the Plumbing Research Laboratory program will, through demonstrations of basic physical principles, be able to clear up much of the disagreement in the plumbing industry over what constitutes safe practice. This, in turn, may lead to some basis for uniform plumbing codes as well as a greater recognition among those in the industry of their responsibility in safeguarding the public health.

Good Practices in Painting Steel Elevated Tanks and Standpipes

James O. Jackson

A contribution to the Journal by James O. Jackson, Pres., Engineering Development Co., Coraopolis, Pa., and Chairman, Committee 8710 J—Joint Com. on Steel Standpipes and Elevated Tanks.

Although the author is chairman of the joint committee, the views expressed in this article are those of the author only and are not to be considered to include the final views of the committee. The painting standard (AWWA D102) is presently undergoing complete revision, and the committee welcomes any suggestions for the improvement of the standard, descriptions of paint failure, or reports of outstanding service from a particular paint system. All such information or suggestions should be directed to James O. Jackson, Chairman, Committee on Steel Standpipes and Elevated Tanks, Box 249, Coraopolis, Pa.

IN order to prolong the life of paint, the surface to be painted should be adequately prepared. This is true for both the interior, or water-immersed, surfaces and for the exterior, or weather-exposed, surfaces. When the steel plates are rolled or dished in fabrication, the mill scale is cracked and left in such a condition that moisture penetrating the paint film may cause corrosion underneath the mill scale, which results in the lifting and breaking, or bulging, of the paint film. The types of paints most resistant to this kind of damage are those that have enough strength and thickness to resist the loosening of the mill scale. Such paints are the heavy tar coatings and the cement linings. The wax or grease coatings are more tolerant to mill scale than the usual paints, because the plasticity of the coatings permits some movement of the mill scale without a breaking of the protective film. It has, however, been well proved that even the heavy tar coatings, cement linings, waxes, and greases last longer and give more economical service if the mill scale is completely removed.

Surface Preparation

If the mill scale is not completely removed, the life of most paint films on either interior or exterior surfaces averages 1–2 years. After this type of failure has occurred, unless the remaining paint and mill scale are completely removed by blast cleaning, usually at considerably higher costs than if it had been done originally in the fabricating shop, the repaint job will be costly and unsatisfactory. Therefore, best practice calls for all mill scale to be removed after fabrication in the shop by either blast cleaning or pickling, and for the shop primer to be applied before rusting takes place.

After the steel has been erected in the field and the field joints have been welded, it is essential that these joints

also be cleaned by blast cleaning to remove the weld slag and spatter from the welded seams, and to remove the rust that may have formed on the unpainted edges of the plates. Any other damaged or abraded places should be blast cleaned and reprimed. The blast cleaning of the welded joints may be done with small, easily portable blasting equipment or special equipment that completely confines the blasting action. It must be emphasized that unless the field seams are blast cleaned and the weld slag completely removed, the life of the field seams will be very much shorter than that of the blast-cleaned plates, and the full benefit of the cost of the surface preparation in the shop will not be realized. If the mill scale is entirely removed in the shop and the field-welded joints blast cleaned, a paint life of 5–6 years may reasonably be expected from paints of good quality, and a life of 8–10 years may be expected from heavy-duty paints. In order to secure such paint service, it is essential that the interior and exterior surfaces be inspected annually and any defects be repaired by the restoration of the original paint systems. Such defects will be rare, but some are almost certain to appear. In repainting tanks that have had a general failure of the paint, it is usually more economical, with regard to the life of the paint job, to remove all the old paint and rust by blast cleaning and to apply one of the paint systems recommended for blast-cleaned surfaces.

It is important that any dust resulting from the blast-cleaning operation be removed from the steel surfaces before the application of the primer, if it is to be applied by spray, for dust can seriously interfere with the adhesion of paint so applied. It is desirable, however, to remove the dust, even if the primer is to be applied by brush.

Interior Surfaces

Paint systems for the interior surfaces of steel water tanks must adhere well to the base metal and to the underlying coats under water, and must have low permeability to water and good resistance to blistering. Coal-tar enamel applied over a suitable prime coat on a properly prepared surface has very low permeability to water and excellent durability under immersion. Such an application, properly put on and maintained, may reasonably be expected to have a useful life of more than 10 years. The initial cost is several times that of a first-quality paint coat.

Sprayed metal coatings, usually either zinc or aluminum, give excellent protection to interior tank surfaces. These coatings must be applied over blast-cleaned surfaces, and a first-class paint system over the sprayed metal surfaces is recommended to decrease porosity and give extended service life. Sprayed metal surfaces cost much more than first-class paint systems, and even considerably more than hot-applied coal tar. Annual maintenance is difficult because of the equipment required to replace the sprayed metal. Usually, maintenance consists only of retouching the painted surfaces.

Cathodic Protection

Cathodic protection is an effective method of reducing corrosion on water tank interiors. This protection requires the installation of anodes and suitable electrical apparatus to maintain a current flow between the anodes and all submerged parts of the tank.

The amount of power required and the initial and maintenance costs of the anodes vary with the electrical conductivity of the stored water and other factors. It is considered economical to paint the interior surfaces even though cathodic protection is used, as such painting reduces considerably the amount of power required, because of the insulating effect of the paint. In latitudes where ice accumulates during the winter, the anodes must be removed. During such periods, the cathodic protection is inoperative. It is imperative that cathodic systems be periodically serviced so that their operation will be continuous and effective. The initial cost of the cathodic system will be considerably more than the cost of a first-class paint job, because the tank with cathodic protection should also be painted, and the paint system should be just as durable as without the cathodic system. The annual cost of the cathodic system, including the protective paint, is therefore considerably greater than that of a first-class paint system. The cathodic system, however, even if the paint system is not maintained, will prevent the pitting of steel for some time.

The most common and, usually, most economical method of protecting the interior surfaces of tanks is with paint systems. The most suitable paint vehicles for water-immersed surfaces are the vinyls and the phenolics. The straight linseed oil vehicles and the alkyd linseed oil combinations are less impervious and more soluble in fresh water than either the vinyls or the phenolics. Other types of synthetic vehicles show considerable promise, such as the epoxy resins, which may be hardened either by chemical reaction or by solvent evaporation, but these paints have not been used sufficiently to justify their recommendations at this time.

Vinyl-Base Paints

Vinyl-paint systems produce underwater paint films that are more durable than those of any other paint systems extensively used on the interior of tank surfaces. A paint life of 8–10 years may be expected from a properly applied vinyl-paint system. The vinyl resins used in metal protective paints are a series of resins made from copolymerized vinyl compounds. The main constituents of vinyl resins are vinyl chloride and vinyl acetate. The copolymer is a relatively inert, relatively insoluble resin of water-white color that has outstanding qualities of durability and resistance. It is soluble only in special types or combinations of solvents. Such vinyl resins adhere poorly to metal. To improve the adhesive properties of the vinyl chloride–vinyl acetate copolymer, other constituents, such as maleic acid, are added. The resulting resins adhere strongly to steel, to other vinyls, or to other paints. These resins are different from the ordinary paint vehicles because they do not oxidize. They dry solely by solvent evaporation, in the same manner as lacquers. Because their usual solvents have low boiling points, the resins are extremely fast drying. Their inertness and fast-drying qualities make them useful paints. Their poor wetting ability, however, limits their use to clean steel. They are used in solutions with low solids content.

The most common vinyl-paint system requires the use of a wash primer pretreatment over a sandblasted sur-

face to provide good adhesion. The vinyl paints have a smaller percentage of nonvolatile ingredients than other common paints, and, therefore, more coats are required to obtain the necessary dry-film thickness. A dry-film thickness of at least 4 mils is considered essential, and even with double spraying for each coat, this usually requires four coats of paint in addition to the wash primer. The vinyl paints dry quickly, and several coats may be applied in a day. Application of this type of vinyl paint by spray is essential, because it dries so fast that it is unsatisfactory for brush applications. Other types of vinyl-paint systems are compounded so that the wash coat pretreatment is not required. Still other vinyl paints incorporate plasticizers and other ingredients to increase the percentage of nonvolatiles, so that the required thickness may be built up with fewer coats. Maintenance of the vinyl-paint systems is not difficult. Any places that show signs of failure may be easily cleaned and restored to the original paint film, owing to the quick-drying characteristics of the paint. It is believed that, because of the long life of the paint film and the little maintenance required, the four-coat vinyl system with wash primer results in the most economical protection for interior tank surfaces.

Phenolic Paints

Phenolic vehicles are very effective in paint systems for underwater service. They produce a film that is quite impervious to water and very resistant to blistering. The most common difficulty with paints based on this vehicle is the tendency for the paint coats to separate or peel, particularly when too much time has elapsed after the application of the underlying coat. This characteristic of the phenolics presents a considerable problem where the steel is primed in the shop with a phenolic primer, because there is ordinarily a lapse of several weeks before the first field coat is applied. One procedure believed to be of value is the addition of a solvent or softening agent to a paint coat after the priming coat has dried for a long time. Two of the most successful solvents are ethylene glycol monoethyl ether * and methyl isobutyl ketone. One recommendation is that 1 per cent of the solvent be added where the paint has dried for 24 hr; 2 per cent, for 48 hr; and 3 per cent, for 72 hr. It can easily be seen that this recommendation does not provide for the condition where the shop primer has dried for several weeks. In spite of the addition of solvent, enough failures have occurred so that it is now considered questionable practice to prime tank plates in the shop with a phenolic primer. The use of the phenolic primer should be limited to instances where it can be applied in the field, so that the subsequent paints can be applied on following days.

The pigments for water-immersed phenolic-vehicle paints include red-lead formulations as inhibitive primers, usually followed by two or three coats of aluminum powder in a suitable phenolic vehicle. The aluminum powder contributes to the imperviousness of the paint film and has given excellent results in underwater service. Zinc dust in a suitable phenolic vehicle is an excellent primer with a sacrificial anodic effect in reducing corrosion. Usually, two coats of the zinc dust

* Cellosolve, a product of Carbide & Carbon Chemicals Co., New York, N.Y.

phenolic primer are followed by one or more coats of zinc dust–zinc oxide phenolic finish coat, as this formulation is believed to be more durable as a finish coat. The zinc dust–zinc oxide formulations may be used both as primers and as finish coats. They produce excellent paint films of good durability, with no problem of lead absorption.

Chromate Paints and Other Coatings

A recently developed pigment known as "M-50"—a basic lead silicochromate pigment—when used with a suitable phenolic varnish vehicle, has shown excellent durability in preliminary tests and will be a valuable addition to paints available for water-immersed surfaces, if the results from actual use prove to be as good as the tests indicate. The basic lead chromate in this formulation is practically insoluble in water.

Other types of coatings are available for the interior surfaces of tanks. Cement linings, troweled on to a thickness of $\frac{1}{8}$–$\frac{3}{16}$ in., usually mixed in a proportion of 1 part cement to 1.5 parts of sand, have provided excellent protection. The strength of the lining undoubtedly resists the tendency of loosening mill scale to break through the coating. For this reason, cement linings are more suitable than paints over surfaces from which the mill scale has not been entirely removed.

Wax and grease coatings are available for water tank interiors. They are applied to a thickness of $\frac{1}{16}$–$\frac{1}{8}$ in. and remain plastic. Because these materials, properly applied, are practically watertight, they give excellent protection. The principal difficulties with this type of protection are those of obtaining a complete unbroken film and the tendency of the films to run in extreme heat and gradually to dry out and become brittle with age. Wax and grease coatings are more suitable than paints that dry over mill scale, because the movement of the mill scale is better tolerated by the more plastic materials.

Exterior or Exposed Surfaces

As with paints for interior surfaces of tanks, it is essential with paints for exterior surfaces to remove mill scale completely, if maximum paint life and economy are to be obtained. It has been the practice in the past to clean the exterior surfaces by removing the loose rust and mill scale with a wire brush and then to apply one shop prime coat of red-lead linseed oil or other paint. After the field joints are welded, they are wire brushed and primed with one coat of paint, after which the exterior surfaces are painted with two or more coats of various finish paints, usually varnish aluminum paints or alkyd finish paints in various colors. Depending on the nature of the mill scale, the thoroughness with which the loose mill scale is removed, and the severity of the exposure, this type of paint system usually starts to break down in about a year and requires complete removal and painting of surfaces in 1–3 years. If the structure is again wire brushed, and the loose portions of the paint are removed and the structure repainted, the experience will be repeated.

Experience over the past decade has demonstrated very conclusively that, to prolong the life of paint economically, the mill scale should be completely removed in the shop before priming. This permits the use of faster drying, more durable synthetic paints. An ex-

cellent paint system for water tank exteriors is a shop coat of alkyd–linseed oil red-lead primer applied over surfaces from which the mill scale has been removed, followed by priming of blast-cleaned field joints with the same paint. Then two coats of aluminum paint, consisting of either a long oil varnish or an alkyd varnish, or two coats of an alkyd-vehicle finish paint of desired color, are applied. The number of coats may be increased to four by the addition of another coat of primer, which results in longer paint life and which is especially desirable for more severe exposures. The three-coat alkyd system over surfaces freed of mill scale should result in a paint life of approximately 5 years, if there is suitable yearly maintenance. The four-coat system should result in about the same paint life, if used where the exposure is more severe.

External vinyl-paint systems over surfaces from which the mill scale has been removed and which have been treated with the wash primer, followed by four coats of vinyl paint to a total dry-film thickness of at least 4 mils, are extremely durable. It is essential that the field welds be blast cleaned and given the same wash coat pretreatment and priming as in the shop. This paint system, properly maintained, should last about 10 years in the average exposure and for a slightly shorter time in severe exposures. Annual costs of alkyd and vinyl systems for exterior surfaces should not be very different over a long period. The vinyl system costs more initially, but the time between complete repaintings will be longer than for alkyd systems. The vinyl system is undoubtedly economical for marine and chemical exposures.

Linseed oil–red-lead primers are excellent over blast-cleaned surfaces for exterior exposures, but have the disadvantage of a long drying time in fabricating plants where drying is not accelerated by baking. Either the long oil varnish or the alkyd aluminum paints are suitable over linseed oil red-lead as finish coats and provide very durable paint systems, provided the linseed oil red-lead is allowed to dry completely.

Many other types of paints are available for exterior finish coats, but care must be used to select paints that have good durability and do not fade excessively.

For additional and more specific information regarding surface preparation, paint systems, and paint application, reference is made to the publications of the Steel Structures Painting Council, 4400 Fifth Ave., Pittsburgh, Pa., particularly to the *Steel Structures Painting Manual*.[1] Volume 1 of the *Manual*, entitled "Good Painting Practice," contains carefully prepared articles on corrosion theory, surface preparation, chemical pretreatment, paint application, inspection, quality control, and comparative costs of paints and paint systems for such applications as railroad and highway bridges, steel vessels, steel tanks, pipelines, water and sewerage works, refineries, and chemical plants. Volume 2, "Systems and Specifications," contains specifications for various types of surface preparation, pretreatment, paint application, specifications for many types of paints, and a complete tabulation of recommended paint systems for various purposes. The paint systems are prepared in such a way that they may be easily incorporated into engineering specifications.

Toxicity From Lead Absorption

For many years, the question of the extent to which lead is absorbed from paints on the interiors of potable-water tanks has been the subject of much discussion. Opinions have varied between those who contend that the lead absorption from paints is of no consequence whatever, and those who will in no circumstance permit the use of paint containing lead on the interior of tanks. Within the past several years, research work has been completed on lead absorption, and two articles on this subject were reported in the May 1959 JOURNAL.

The laboratory investigation made by Fraser and Fairhall [2] of USPHS brought out the important fact that the lead in linseed oil–base paints is "at least ten times more soluble under the conditions of the test than either the phenolic- or vinyl-base paint." The oil-base paint tested did not contain added litharge. It was further concluded by them that the characteristics of the waters found in different localities has a profound effect on the rate of solution of red lead from paint films.

Elkins [3] reported on the results of many analyses of the lead content of water from actual storage tanks:

> A study of the lead content of water from eleven standpipes painted with lead paint has been made. Of 70 samples of water, only three contained significant amounts of lead, and these were from tanks in standby service. Tests of the extraction of lead from panels coated with red-lead oil paint indicate that no significant quantity of lead is added to the average water supply by the paint if the average daily consumption approaches the capacity of the tank; and if a phenolic-base paint with a red-lead pigment is used, a substantially lower rate of turnover is essential to keep the lead concentration below significant levels.
>
> All the evidence available indicates that only under extremely exceptional conditions, if at all, could the use of lead paint in standpipes for potable water be considered inadvisable from the standpoint of public health.

Neither article has adequately answered many questions with respect to lead absorption. For example, no information was given about the contribution of lead to drinking water from underlying paint coats containing lead. Little is known about the relative absorption for such variables as the various types of stored water, the length of time water is retained in tanks, average storage temperatures, pumping cycles, and many other variables.

The Joint Committee on Steel Standpipes and Elevated Tanks, at the Dallas meeting of AWWA in 1958, recommended that full-scale research be conducted on painted tanks in the United States to obtain lead measurements and information on the important variables. This project was approved by the AWWA Executive Committee, but had to be abandoned later because financing could not be obtained. A ballot was sent to the members of the committee to determine the recommendation on the use of paints containing lead on the interior of potable-water tanks. The outcome was that lead paints having phenolic or vinyl vehicles should be recommended as final coats, but that lead paints containing linseed oil vehicles should not be recommended because of the much higher rate of solubility. The committee's decision is not an indication that it believes serious health hazards have been or would be caused by the use

of linseed oil lead paints as final coats in water tanks, insofar as the contribution of lead from the paint alone is concerned. The committee's decision was based on the realization that any additional amount of lead absorption is undesirable, inasmuch as the present daily average intake of lead is close to the maximum tolerance. Further, the committee is of the opinion that equally suitable, or even more suitable, paints are available, containing either lead in combination with vehicles that do not permit as high an absorption rate, or containing no lead at all.

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R906	EASTMAN	Bond Issue Campaigns	Sep 59	16	25¢
R907	INGRAM & MOORE	Fluoridation in Major Cities	Sep 59	16	25¢
R910	PIPER	Model Water Law	Oct 59	6	20¢
R911	FAUST	Plastic Pipe Standards	Oct 59	4	15¢

Advances in Water Supply Technology

A. A. Kalinske

A paper presented on Sep. 29, 1959, at the Missouri Section Meeting, Kansas City, Mo., by A. A. Kalinske, Vice-Pres. & Tech. Director, Infilco Inc., Tucson, Ariz.

MANY processes and techniques in water treatment have not changed much for 50 years or more. It is difficult to imagine that the early practitioners and designers in the field of water supply engineering were so technically advanced and farsighted that their efforts could not be improved on for half a century.

Some of the reasons for the apparent lack of progress, or slow progress, in the water supply field may be:

1. Inadequate fundamental research in the universities and government laboratories (This situation, fortunately, has been improving during the last several years.)

2. Promulgation of old techniques and standards by regulatory agencies, and slowness of some of those charged with enforcement to accept new ideas, or even to provide means for the testing of new techniques and equipment

3. Lack of enough interest, on the part of many consulting engineers, to study and look at new techniques and equipment until someone else has done the pioneering.

The attitude prevalent in the water supply field—more so, perhaps, than in any other—seems to be: "we will consider it after you can show us at least half a dozen installations," or, "give us several successful reference installations and then you can do it in our state." This last criticism should not be misinterpreted, for regulatory agencies should, of course, take a conservative attitude toward new techniques and equipment, as they have the all-important responsibility of protecting the public health and seeing to it that the public funds are used judiciously. Engineers should, of course, be skeptical of, and impartially evaluate, anything new. The criticism, however, is neither of conservatism nor of good, sound engineering caution; rather, it is directed at the lack of a framework within which a new technique can be evaluated and the fruitless evasiveness of always relying on someone else to try it first.

Solids-Contact Process

In the treatment of water before filtration, the outstanding development during the last decade has been the greatly increased use of the solids-contact process, not only for softening but also for clarification of surface water. The increase in the popularity of this process is unquestionably due to the fact that such pretreatment both affords significant savings in first costs and produces a high-quality effluent for filtration. Effluent turbidities of such basins are commonly of the order of 5 ppm and rarely, if ever, exceed 10 ppm. It is also significant that solids-contact units are being used for larger and larger treat-

ment plants. Single units of 30-mgd capacity, which incorporate mixing, flocculation, and clarification, have been built.

In a paper [1] presented before the sanitary engineering division of the ASCE in October 1958, Richard Hazen compared data on turbidity removals at conventional and solids-contact treatment plants handling river waters in the eastern United States. He indicated that good turbidity removals were obtained at surface loadings varying from 1.2 to 2.0 gpm/sq ft without the use of any coagulant aids. He stated that such good performance was apparently due to the better flocculation obtained in solids-contact treatment units.

Even though hundreds of solids-contact treatment units are used in the United States, there are still some states that drastically restrict the use of such treatment units, and arbitrarily set up standards that make their use entirely uneconomical. In fact, several states have no solids-contact treatment units on municipal supplies, even though there may be dozens of such units in adjacent states. Solids-contact units must, of course, be specifically designed for each particular water treatment problem, and no general rules can cover each and every situation. Many states feel that it is necessary to maintain certain very strict standards for retention time and surface loading. The regulations are flexible, however, in that if scientific evidence is presented conclusively indicating that shorter retention times and higher surface loadings can be used safely, the consulting engineer may present such data to the state department of health for consideration. If the data justify it, the engineer may be permitted to design the treatment units as he proposes. Missouri has such a regulation, which, in the author's opinion, is very satisfactory and provides proper protection to the public. The regulation also makes the consulting engineer responsible for gathering sufficient data and insures that the treatment plant is designed scientifically and not merely on the basis of general guides and rule-of-thumb criteria.

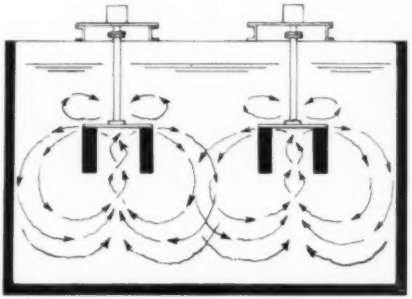

Fig. 1. Agitation Pattern of Turbine Flocculator

Turbine flocculators are low-speed centrifugal pumping units which move large quantities of water throughout the basin and produce much more complete agitation than the conventional paddle stirrer.

Flocculation Equipment

Probably, no piece of equipment in general use in conventional treatment plants has undergone fewer changes during the last several decades than the paddle-type of flocculator. Studies of the hydraulic conditions that should be obtained in a flocculation basin have shown that paddle stirrers are not necessarily the best flocculating equipment that can be used. The principles of good flocculation are well established. It is known that turbulence must be so controlled that the precipitated floc can repeatedly come into contact with other floc particles without any shear-

ing or breaking of the agglomerated floc.

The turbine type of agitator, so widely used for mixing operations and in solids-contact units, has application in conventional flocculation basins (Fig. 1). Radial-flow turbines can be installed on vertical shafts, thus making underwater bearings unnecessary, or in long rectangular basins they can be installed on a horizontal shaft, with the requisite number of piers and bearings. Hazen [1] indicated that turbine flocculators are more efficient because they eliminate the relatively dead space along the axis of conventional paddle stirrers. He stated that turbine flocculators produce a more uniform mixing action throughout the flocculating chamber.

Turbine flocculation equipment may, offhand, appear entirely inadequate for producing agitation and turbulence throughout the basin. That impression is based on the faulty notion that the turbines are mere stirrers, which they definitely are not. Turbine flocculators are low-speed centrifugal pumping machines which move tremendous quantities of water throughout the basin, generating the low-intensity turbulence which is so effective in producing proper contacting of the floc particles without exerting excessive shearing action.

Chemical Pretreatment

So far as the chemical treatment of water before filtration is concerned, there have been several significant developments during the past decade. Undoubtedly, the most significant development has been in the use of coagulant aids. The pioneering work of Baylis [2] in connection with activated silica indicated that one need not put up with poor flocculation, with the resultant poor quality of water produced for filtration. Activated silica is now widely used as a coagulant aid, and, as a result, waters which formerly produced very poor settling floc can now be treated and clarified in settling basins of much smaller size than would have been necessary had activated silica not been used. Activated silica is also being used fairly extensively as the sole coagulant in softening treatment. In many instances, it has been found more economical than alum.

Polyelectrolyte Coagulant Aids

More recent developments in coagulant aids have been the polyelectrolytes and other organic coagulant aids. There is now a wide variety of such compounds, and several of them have been approved as nontoxic and can be used for treating potable water. At present, many of the coagulant aids are still quite expensive; however, their use should be evaluated and definite information should be obtained as to how much the settling characteristics of the floc are improved, and just how much smaller clarification basins can be made. Coagulant aids are more widely used for industrial water treatment than they are for treatment of public supplies, where operating costs must be given very serious consideration at all times. So far as the author has been able to determine, no general statements can be made regarding the effectiveness of any of these coagulant aids for any particular water treatment problem. Each one should be studied individually under various conditions to establish what improvements in floc characteristics it can produce.

Coagulation Research

The water supply field can look forward to a more complete understand-

ing of the general process of coagulation in the future. Coagulation of colloidal turbidity with alum or an iron coagulant is still, to a large degree, a trial-and-error proposition and cannot really be called an exact science. Some very outstanding basic research on coagulation is now in progress under the direction of A. P. Black at the University of Florida. The project is sponsored by the National Institutes of Health of USPHS. Progress reports on this work have already been published in the JOURNAL.[3] It is just such basic research that will, in years to come, form the foundation for new processes, techniques, and equipment. The importance of such studies cannot be overemphasized, and it is to be hoped that more will be undertaken.

Filter Design

There has been no significant change in rapid sand filtration during the last 40 years. Gradually higher filtration rates have been used; this is certainly justified, if somewhat belated, in view of improvements in the quality of water now being applied to filters as a result of improvements in pretreatment. The most expensive parts of a water treatment plant are the gravity filters and their appurtenances. In terms of their effect on water quality, however, gravity filters do a very small portion of the work; in many places, in fact, gravity filters have become a mere safety device to prevent the passage of water of poor quality through the treatment plant if something should go wrong with the pretreatment facilities.

Hudson [4] and others have presented evidence that it is not necessary to operate filters at a preset constant rate, but that the filtration rate can vary from a relatively high initial rate, when the filter is clean, to a lower rate as the filter becomes clogged and dirty. In other words, expensive control equipment designed to maintain a uniform flow through the filter, regardless of its clogging condition, may really not be necessary. There is evidence that the great refinements that have been built into controllers to maintain a uniform filtration rate are not scientifically justified, so far as water quality is concerned. Hartung [5] has demonstrated that filters can be operated satisfactorily and produce a water of high quality without controllers. It is not the least bit unlikely that future gravity filters will be designed for higher operating rates, and that the attendant control equipment will be either greatly simplified or altogether eliminated.

Backwashing of Filters

Probably no other issue is the subject of more widespread controversy among engineers than the issue of the best design of the underdrain system of gravity filters to secure a uniform and proper backwash distribution. Through the years, a great variety of underdrain systems has been developed. Unfortunately, fundamental hydraulics (which, in this instance, are far from simple) have been neglected or incompletely understood. Peculiarly enough, very little basic research has been done in connection with the hydraulics of underdrain systems and flow through granular media. The author has been able to find only one good basic analysis of the hydraulics of a filter underdrain system, and that was published only 5 years ago.[6]

In travels in various European and Latin American countries, the author has noted that wherever European engineers have been influential the air-water backwash system is used. This type of backwashing certainly seems to do an excellent job of cleaning the sand. According to the engineers and

operators who are familiar with this air-water backwash system, troubles such as shifting gravel beds, mudballs, and upset sand beds are practically unknown. Such difficulties are quite common in the United States, however, and plague many filter plants. It appears that these troubles are, to a great extent, related to the particular system of backwashing that is used in the United States—that is, high backwash rates, which may at times be high enough locally to upset and shift the supporting gravel.

In the European air-water backwash system, the water wash rate is only about half of the rate usually used in the United States. Good scrubbing of the sand is accomplished by the air, and not by high-velocity water. The agitation and turbulence produced by a combination of air and water appears to be much greater than that produced by a high-velocity water backwash alone. It thus appears that Europe is somewhat advanced over the United States in the field of backwash design and practice. Why this has taken place is a difficult question to answer, in view of the pride Americans take in the progress they have made in sanitary engineering. This, the author believes, is an outstanding example of what happens when everyone is satisfied to continue using standards and regulations that have been set up in years past, and when no one is concerned with making research studies to establish definitely whether certain other methods and techniques are better.

Probably one reason why there has been very little research in connection with filter backwashing is that there is no particular incentive for an equipment manufacturer to carry on such studies; there does not apear to be much that he could sell from an equipment standpoint. The author understands that USPHS has set up a research project to study the hydraulics of backwashing, including the air-water backwash, and that this research is to be carried on at the California Institute of Technology.

Diatomaceous-Earth Filtration

In recent years several small treatment plants have been using diatomaceous-earth filters. Baumann [7] reported in 1957 that 30 municipal plants in the United States were using diatomaceous-earth filtration. These plants treated water of relatively low turbidity, requiring, in most instances, no pretreatment. Because of the very small first cost, diatomaceous-earth filters may, in the future, replace many gravity sand filters. Again, progress in that direction may be hampered, or even prevented, by state regulations that have outlived their usefulness.

The removal of alum or iron floc by diatomaceous-earth filtration is something that cannot be looked forward to with too much confidence. Even relatively small amounts of alum floc tend to clog a diatomaceous-earth filter very rapidly. Pretreatment of alum-coagulated waters must be carefully done, and the amount of floc entering the filter must be kept very small (perhaps 2–3 ppm). Also, filtration rates must be reasonable, and certainly should not exceed 2 gpm/sq ft. In many instances, even 1 gpm/sq ft should not be exceeded if a reasonable length of run is to be obtained. Continuous feeding of diatomaceous-earth is quite important if a relatively large amount of suspended matter is to be removed, and if the length of run is to be of at least 10–12 hr. This form of filtration has an application in the field of potable water treatment, but it cannot be applied widely. Until a great deal more information has been

accumulated, each treatment problem will have to be studied individually, and no general rules can be established.

Disinfection

So far as disinfection is concerned, there is no question that the United States has gone very far in producing safe water supplies. This does not mean, however, that there is no need for further research. No one can afford to overlook the possible applications of ozone, which is so widely used in Europe. Bacteriologists and biologists still have much to learn about what is needed for insuring the destruction of all filtrable viruses.

Saline Water and Reclamation

In the future, there will probably be more and more reuse of water, both in industry and for domestic purposes. This will be particularly true in areas where a suitable natural source is practically unobtainable. It will also be quite interesting to follow the developments in desalinization of sea water and brackish water. Some of the desalinization techniques and equipment that have recently been developed are really phenomenal, when compared to what was possible, both technically and economically, 5–10 years ago. It is not improbable that new distillation techniques, or some of the other methods for removing dissolved solids, may become so economical that chemical treatment and filtration of turbid waters may become outmoded in some areas. This most certainly will be true if cheap atomic power becomes available.

Summary

There is definite evidence that research in various fields of water supply engineering will increase in the future, and many more advances can be expected through government-supported research. New developments, both in techniques and in the equipment field, may be accelerated by increased interest on the part of equipment manufacturers, if engineers and regulatory agencies take more realistic attitudes toward new processes and equipment.

There is a definite trend to cut the costs of water treatment plants, and to design each plant for the specific job it is to do. Diatomaceous-earth filtration may become more popular, especially for smaller plants, in order to reduce present high costs of construction.

New developments in the reclamation of used water, both in industry and for domestic use, may open up new fields for research. Saline water conversion, particularly if a source of cheap power can be found, may one day altogether change the face of water supply practices.

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Test Program for Filter Evaluation at Hanford

Walter R. Conley and Raymond W. Pitman

A revision of a paper presented on Apr. 24, 1959, at the Pacific Northwest Section Meeting, Vancouver, B.C., by Walter R. Conley, Engr., and Raymond W. Pitman, Specialist, both of the General Electric Co., Richland, Wash.

A FILTER evaluation program, in which small experimental filters are utilized, has been conducted at Hanford, Wash., since 1950. The program became most useful when a 5-gpm experimental plant was constructed and operated at Hanford in 1955. The test data presented have been confirmed in the production plants at Hanford. It is believed that the experimental results may be of use to water utility engineers and operators.

Equipment

A variety of arrangements of test filter equipment is used in the program. The smallest filters are approximately ¾ in. in diameter and 2 ft long. These small filters are used to obtain rough approximations of the effect of changing water treatment variables. The water to be filtered is treated in 4-liter beakers. The value of using the small filters is the speed with which useful data can be obtained. The limitation of the filters is that the volume of water treated and the size of the filters are both so small that misleading results are sometimes obtained. Evidence or data from the use of these small filters are considered only indicative of the probable results that will be obtained with the use of larger filters.

A second type of filter that has been used consists of a glass tube about 2.5 in. in diameter and 6 ft long. This type was used to filter water treated in the full-size filter plants. Flow was controlled at constant rates of 2, 4, and 6 gpm/sq ft. The test filters were packed with a variety of materials, and data on head loss and water quality were obtained. These filters were found to be reliable, but they were of limited value because pretreatment of water could not be varied as desired, owing to the necessity of maintaining uniform standard operations in the production plants. An additional limitation was the slow and rather inaccurate methods of measuring filtered-water quality then in use, methods that consisted of aluminum analyses, iron analyses, and turbidity analyses with a Baylis turbidimeter.

Thus far, the most satisfactory equipment used for filter evaluation is a small 5-gpm complete filter plant (Fig. 1 and 2). This plant is equipped so that during chemical treatment, flocculation time and speed, settling time, and filtration rate can be varied over a wide range. Data from this small plant have proved to be very reliable. After about 4 years of testing, there have been no instances of substantial differences between data obtained in

the experimental plant and data obtained in the operational plants.

Another test filter is a full-size plant filter, used to demonstrate the validity of the data obtained from the smaller filters.

Since 1954, a light-scattering microphotometer (Fig. 3), modified for the specific purpose of measuring filtered-water turbidity, has been used at Hanford.[1] This instrument makes it possible to evaluate filter performance more accurately and sensitively than with other equipment. Results of the test program show that the measuring instrument is essential if fundamental data on filters are to be obtained quickly.

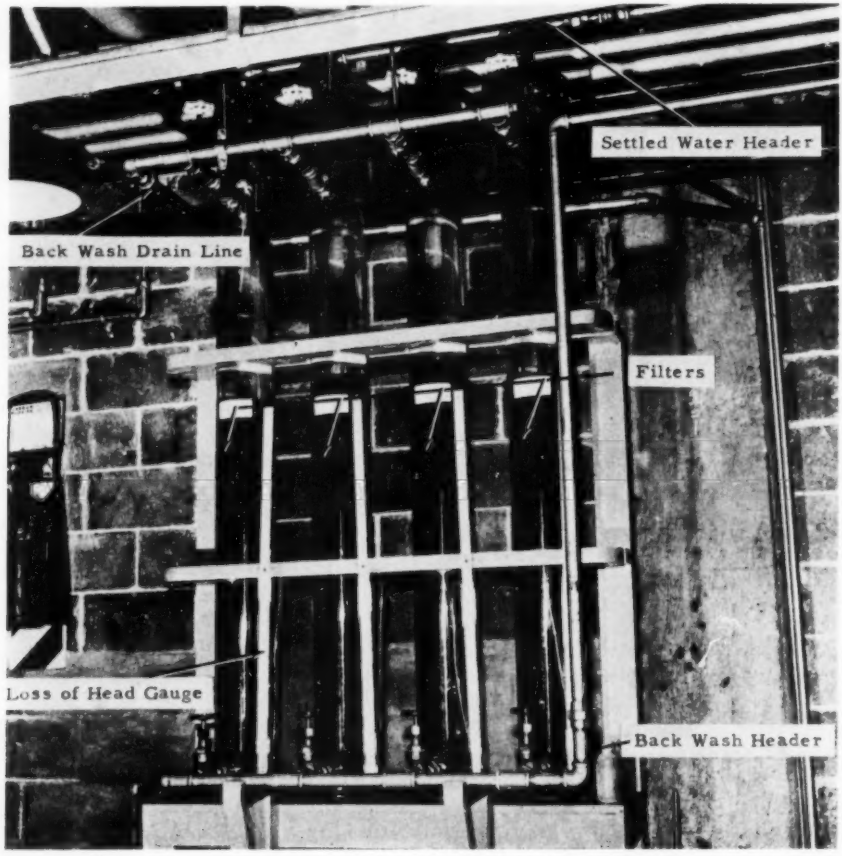

Fig. 1. Filter Setup at Experimental Plant

The settled water passes through a 2-in. pipe into the filters, which are 6 in. in diameter and loaded with filter media to the same depth as in the standard Hanford production plant filters.

Criteria

The following criteria are used for evaluation of filter performance:

1. The filter must produce filtered water containing less than 0.01 ppm turbidity for all raw-water conditions (assuming optimum treatment), at a rate of 6 gpm/sq ft and a settling time of 1 hr.

2. The duration of the filter run must be at least 6 hr, at a filtration rate of 6 gpm/sq ft and a settling time of 1 hr for the worst raw-water conditions encountered. This gives a rate of head-loss increase of not more than 1 ft/hr.

The specified minimum duration of a filter run is based on practical considerations: manpower required for filter washing, water lost while the filter is off the line, and water used for the backwash operation. A duration of 6 hr is not the absolute minimum from this standpoint, but it is the most practical.

Sand Filters

Filters made of sand only, of various sizes, were checked against the foregoing criteria. Typical results, when the alum feed is fairly small (10 ppm) and breakthrough is not a problem, are shown in Table 1. It will be noted that the finer sand produces an effluent of greater clarity than that produced by the coarser sand. Additional experiments showed that the difference in clarity among the sands could be reduced by feeding excess amounts of alum. Typical results with a relatively large alum feed (20 ppm), when breakthrough is not a problem, are also shown in Table 1.

It will be noted that the difference in effluent clarity is small when alum feed is high—that is, filters made of coarse sand require more alum than filters made of fine sand. Although the data indicate that some of the sands meet the proposed criteria, this is not true for all situations. For example, when the raw water is difficult and coagulants, such as activated silica or other chemicals, have to be used to control filter breakthrough, all the filters fail to meet the criteria. The Hanford method of controlling breakthrough is to add a few parts per billion of a polyelectrolyte coagulant aid

TABLE 1

Performance of Sand Filters With Different Alum Feeds, at Filtration Rate of 6 gpm/sq ft

Sand Size US Sieve*	10-ppm Alum Feed		20-ppm Alum Feed	
	Effluent Turbidity *ppm*	Head Loss† *ft/hr*	Effluent Turbidity *ppm*	Head Loss† *ft/hr*
20–30	0.008	0.4	0.003	0.3
30–40	0.003	0.6	0.001	0.5
40–50	0.002	0.9	0.001	0.9
50–60	0.001	1.5	0.001	1.5

* Passing–retained sizes are shown.
† Head loss, of course, changes with the season of the year, and can be changed by filter design.

TABLE 2

*Performance of Sand Filters Under Breakthrough Conditions, With Optimum Treatment**

Sand Size US Sieve†	Effluent Turbidity—*ppm*		Head Loss *ft/hr*	Filter Conditioner Feed *ppm*
	Start of Run	End of Run		
20–30	0.005	0.1+	2.5	0.06
30–40	0.004	0.005	2.0	0.03
40–50	0.003	0.003	2.0	0.02
50–60	0.002	0.002	2.5	0.01

* A filtration rate of 6 gpm/sq ft was used.
† Passing–retained sizes are shown.

directly to the water entering the filters. This process is called "filter conditioning." These organic "filter conditioner" chemicals cause the floc particles to adhere tenaciously to the filter media. The method has proved to be superior to all other known methods in controlling breakthrough. Typical results are shown in Table 2.

The high head loss on the coarser sand filters is caused by the need to feed large amounts of filter condition-

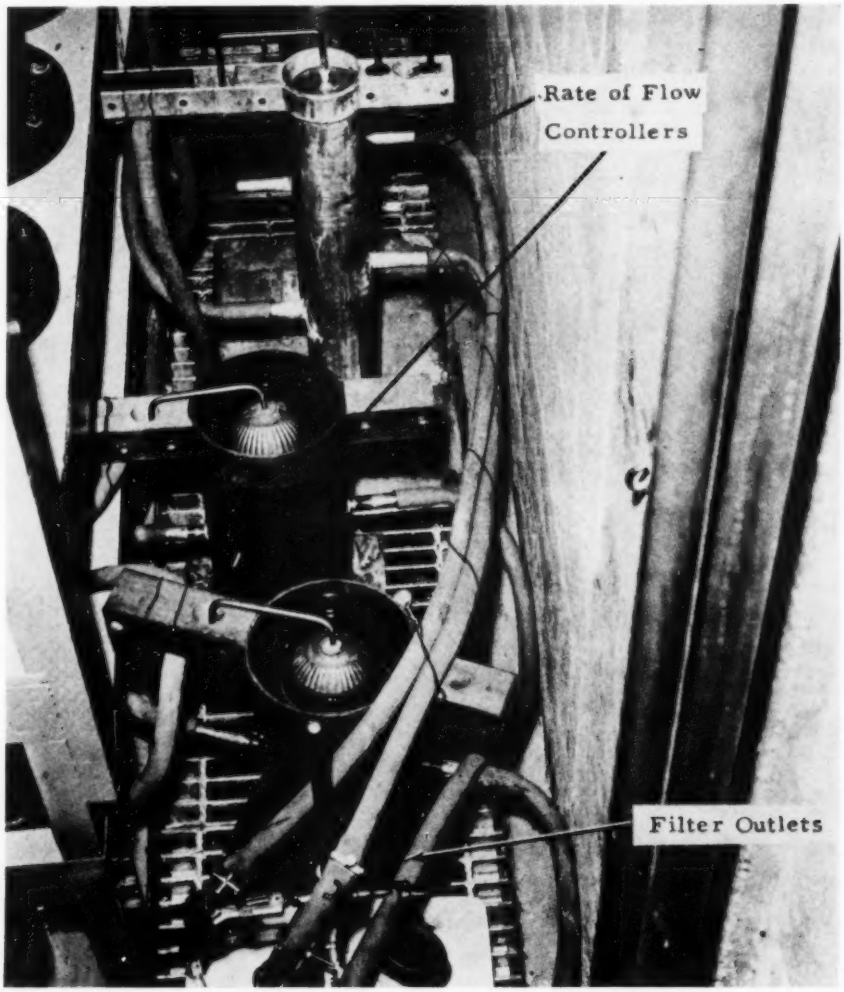

Fig. 2. Rate-of-Flow Controllers at Experimental Plant

The rate-of-flow controllers automatically compensate for the progressive change in filter bed friction loss as coagulated matter clogs the filter.

ers in order to control breakthrough. The fine sands gave exceptionally clear filtered water, but the head loss was very high.

Anthracite Filters

Filters made of anthracite of various sizes were checked against the proposed criteria. Typical results were very similar to those obtained with sand of the same mesh size. None of the filters met the criteria.

Sand-Anthracite Filters

Filters made of sand and anthracite together were checked against the criteria. Typical results are shown in Table 3. Treatment conditions were the same for all filters (low alum feed and no conditioner feed). The table shows that water clarity from all filters is essentially the same, with greater sand depth and smaller sand size giving somewhat better results. Head losses on the filters, however, are widely different, but all are very low by the usual standards.

The kind of raw water being treated and the kind of treatment have a marked influence on the results obtained from all the filters. For this reason, comparisons were made of filter performance during ideal treatment conditions for the filter composition used. For example, when coarse sand is used, the alum feed and filter conditioner feed must be fairly high. When fine sand is used the alum and filter conditioner feed should be fairly low. Table 4 shows the typical results obtained when chemical treatment was optimum for the filters tested. Raw-water conditions were difficult, comparable to those for the filters in Table 2.

As shown in the table, the differences in filtered-water clarity are small at the start of the filter run, but the filters made with coarse sand or entirely of coarse anthracite break through badly at the end of the filter cycle. Filters made of coarser materials require more alum and more filter conditioner feed than the filters made of finer materials.

The performance of the filters with regard to head loss is surprising. The filter made of coarse sand and anthracite (Filter 1) requires so much filter conditioner that the head loss is actually higher than for any of the other filters. Filter 5, made of anthracite with no sand, has the next highest head loss. In spite of optimum filter conditioner feed, these two filters break through badly near the end of the filter run. The general pattern shows that the fine sand is of basic importance in preventing filter breakthrough and insuring good filtered-water quality. The anthracite is of basic importance in reducing filter head loss. Filter 3, made of fine sand and very coarse anthracite, has the lowest head loss of any of the filters, and its filtered-water

TABLE 3

*Performance of Sand-Anthracite Filters Under Easy Treatment Conditions**

Sand		Anthracite		Effluent Turbidity *ppm*	Head Loss *ft/hr*
Size US Sieve†	Depth *in.*	Size US Sieve†	Depth *in.*		
30–40	3	10–40	27	0.003	0.25
30–40	6	10–40	24	0.003	0.30
30–40	10	10–40	20	0.003	0.35
30–40	20	10–40	10	0.002	0.40
30–40	6	10–30	24	0.003	0.20
30–40	6	10–20	24	0.003	0.15
30–40	8	6–18	24	0.004	0.10
40–50	6	10–30	24	0.002	0.30

* Low alum feed, no conditioner feed. The filtration rate was 6 gpm/sq ft.
† Passing–retained sizes are shown.

clarity is almost as good as that for the best filter. Filters 2, 3, and 4 meet the proposed criteria.

Head Loss Distribution

Tests were made of the head loss distribution at various depths in the filter beds. Typical results for difficult treatment conditions at the end of filter runs are shown in Table 5. These data show that floc particles penetrate deeply into the filter beds. Although the sand particles have a greater specific gravity than the anthracite particles, the difference in size of the particles leads to substantial mixing of the materials after backwash. Sand particles were observed as high as 12 in. up into the anthracite on the filter made of very coarse anthracite and fine sand. The mixing at the junction of the sand and anthracite exerts a very favorable influence on head loss, because the finer sand grains cannot form an impervious mat when mixed with coarse anthracite. This effect is found in Filter C of Table 5. Filtration is more or less uniform throughout the entire bed. The coagulated matter is accumulated throughout the filter rather than at the surface of the sand and anthracite. The end result is longer filter runs than can be obtained with any of the other filters.

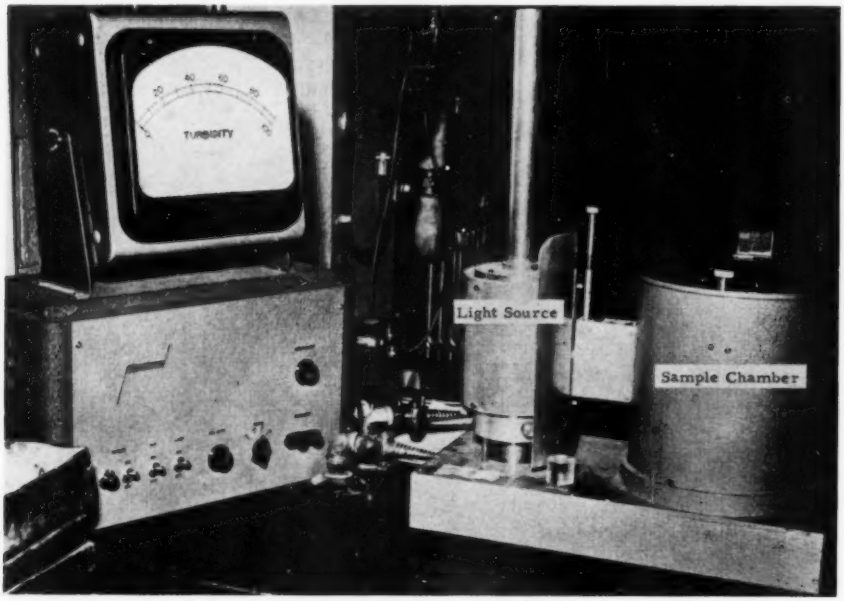

Fig. 3. Light-Scattering Microphotometer

The filtered water is analyzed on the microphotometer at regular intervals. Filter breakthrough or poor filter recovery is easily and quickly detected.

Distribution of material in a given filter bed varies greatly, depending on

TABLE 4

*Performance of Filters Under Breakthrough Conditions With Optimum Treatment**

Filter	Sand		Anthracite		Alum Feed *ppm*	Filter Conditioner Feed *ppm*	Head Loss *ft/hr*	Filtrate Turbidity	
	Size US Sieve†	Depth *in.*	Size US Sieve†	Depth *in.*				Start of Run *ppm*	End of Run *ppm*
1	20–30	8	10–40	22	12	0.06	1.2	0.005	0.1+
2	30–40	6	10–40	24	10	0.03	1.0	0.004	0.005
3	30–40	8	6–18	22	10	0.05	0.7	0.004	0.005
4	30–50	6	10–30	24	9	0.03	0.9	0.003	0.004
5			10–40	30	18	0.06	1.1	0.006	0.1+

* Filtration rate, 6 gpm/sq ft.
† Passing–retained sizes are shown.

the raw water and the treatment conditions. For example, when filter conditioner feed is increased beyond the optimum point, the top surface of the bed does almost all the work. Similarly, when the raw water contains substances that promote the condition known as strong coagulation, the top surface of the filter does a very high percentage of the work.

Filter conditioning allows precise control of floc penetration into the filter bed and, therefore, precise control of water quality and the length of the filter runs.

Turbidity Analysis

All the turbidity determinations noted were made on a modified light-scattering microphotometer.[1] This instrument is so sensitive that substantial amounts of turbidity are observed in distilled water. The accurate calibration of this instrument in terms of the *Standard Methods*[2] turbidity standards is virtually impossible, but the instrument can be calibrated to give good relative turbidity values. The instrument was compared with the Baylis turbidimeter. Some typical results that illustrate the sensitivity of the microphotometer are shown in Table 6.

Calibration of the microphotometer is complicated by light distortion by the sample cell and by contamination of the standard samples as material is removed from the sample container by the solvent action of the water. Calibration proved so difficult that the final method chosen was as follows: Average suspended matter in filtered water was determined by the cotton plug method of Baylis. At the same time, samples of this water were run on the

TABLE 5

Head Loss Distribution at Various Depths in Sand-Anthracite Filter Beds

Depth in Filter *in.*	Percentage of Total Pressure Drop		
	Filter A*	Filter B†	Filter C‡
2	40	20	10
10	55	30	20
20	57	60	70
24	75	85	90
30	100	100	100

* Anthracite size (US sieve), 10–40; depth, 24 in. Sand size (US sieve), 30–40; depth, 6 in.
† Anthracite size (US sieve), 10–20; depth, 24 in. Sand size (US sieve), 30–40; depth, 6 in.
‡ Anthracite size (US sieve), 6–18; depth, 22 in. Sand size (US sieve), 30–40; depth, 8 in.

microphotometer, and the percentage galvanometer deflection was noted. After a period of several weeks, a correlation was obtained between the cotton plug data and the microphotometer readings. A colloidal silica solution * was then used to reproduce this set point. At a later date, bentonite solutions were used to replace the colloidal silica. This calibration method was standardized, despite the recognized weakness of the procedure. It is thought that the absolute turbidity values obtained by the Baylis plug method are low because some of the particles pass through the cotton. For this same reason, the microphotometer readings are believed to be low. In spite of this, the high sensitivity of the microphotometer and its reproducibility are beyond question. The readings should be considered relative, rather than absolute.

TABLE 6

Comparison of Turbidimeter Readings

Sample*	Microphotometer	Baylis Turbidimeter
Distilled water (1)	0.02	0.0
Triple-distilled water (2)	0.001	0.0
Plant filtered water (1)	0.005	0.0
Plant filtered water (2)	0.01	0.0
Plant filtered water (3)	0.02	0.0

* The numbers in parentheses refer to different samples; that is, there were two samples of distilled water and three samples of filtered water.

Filtration Rate Effect

When water treatment conditions are difficult, the filtration rate has a decided effect on filtered-water quality, unless corrective action is taken. The most effective corrective action found in the Hanford laboratories is the use of filter-conditioning agents. Data were obtained at filtration rates from 2 gpm/sq ft to 35 gpm/sq ft. Typical results are shown in Table 7.

The data indicate that when no filter conditioner feed is used, the turbidity of the effluent water increases as the rate increases. But when the filter conditioner feed is adjusted properly, the filter flow has no effect on water quality, at least for rates as high as 35 gpm/sq ft. The increase in head loss, however, at the higher flow rates is very high. It has not been possible, with the knowledge available today, to increase the filtration rates to more than 8–10 gpm/sq ft, because of excessive head loss. From a water quality standpoint, rates much higher than this are satisfactory.

Conclusions

After testing these filters, as well as numerous other types of filters, and evaluating the results against the proposed criteria, a number of conclusions (when treating Columbia River water) have been reached:

1. Filters made either of sand or of anthracite alone do not meet the proposed criteria.

2. Filters made of sand and anthracite together meet the proposed criteria when filter conditioning is used.

3. Sand that passes a US No. 30 sieve and is retained on a US No. 40 sieve is optimum for a sand-anthracite filter.

4. Anthracite that passes a US No. 6 sieve and is retained on a US No. 18 sieve is optimum for a sand-anthracite filter.

5. Acceptable depths for the materials are: 8 in. for the specified sand

* Ludox, manufactured by du Pont de Nemours & Co., Wilmington, Del.

and a 22-in. depth for the specified anthracite.

The experimental results presented in this article were obtained from studies of Columbia River water. This water contains relatively small amounts of organic matter. The turbidity of the river is usually relatively low, but has been more than 1,500 ppm for short periods of time. Peak values of 100–200 ppm turbidity are typical. Algae and other organisms in the water are not present in sufficient numbers to cause a major problem. Results obtained with other kinds of water will be different from the results obtained with Columbia River water; the degree of the difference may be very small or very large.

TABLE 7

Effect of Filtration Rate on Treatment Requirements

Filtration Rate *gpm/sq ft*	Filter Conditioner Feed *ppm*	Filtrate Turbidity at Start of Run *ppm*
2	0.00	0.005
6	0.00	0.01
10	0.00	0.03
15	0.00	0.1
2	0.00	0.005
6	0.03	0.005
10	0.05	0.005
15	0.07	0.005
35	0.15	0.005

The criteria used for judging filter performance are subject to debate. The effluent turbidity requirement may be too stringent. If these criteria are changed, some of the conclusions and the composition of the filters will have to be changed. Furthermore, the complexity of the subject and the numerous variables involved make it likely that some of the conclusions reached will have to be modified to some degree after more data are obtained.

In spite of these limitations, it is believed that the data and conclusions are of general usefulness and can be applied, at least to some degree, in any filter plant operation. Such application should be cautious and experimental.

Summary

Filters made of sand alone, anthracite alone, and sand and anthracite together, of various particle sizes and depths, were evaluated in experimental facilities at Hanford. As an aid to judging the filter performance, criteria were prepared which specify that a satisfactory filter must produce filtered water containing less than 0.01 ppm turbidity and that the filter run must be at least 6 hr for all raw-water conditions encountered, at a filtration rate of 6 gpm/sq ft and with a settling time of 1 hr.

Filters made either of sand alone or of anthracite alone did not meet the criteria. The filters failed either to meet the quality requirements or the length-of-run requirement. But a number of filters made of sand and anthracite together did meet the criteria. Filters made of the two materials were studied extensively with respect to various particle sizes and depths. The best combination was 8 in. of 30–40 mesh sand (US sieve) and 22 in. of 6–18 mesh anthracite.

Performance of all filters varied greatly with chemical treatment and raw-water changes. Filter evaluation must include these variables.

Distribution of coagulated matter at various depths in the filter bed was studied. It was shown that long filter

runs depend on reasonably uniform distribution of coagulated matter throughout the depth of the filter. This uniform distribution can be approximated by careful selection of filter media and judicious chemical treatment.

Generally, carefully made filters can be operated at any filtration rate desired to obtain filtered water of 0.01 ppm turbidity or less (assuming the use of special chemical treatment). The practical limitation on the filtration rate is not quality of water but length of filter run. For Columbia River water, the upper limit of the filtration rate is nearly 10 gpm/sq ft.

Acknowledgment

The studies made were strongly influenced by the fundamental work of J. R. Baylis and associates. Much of the Hanford equipment design, such as the test filter, cotton plug filters, and the turbidimeter, originated with Baylis.

References

1. Conley, W. R. & Pitman, R. W. Microphotometer Turbidity Analysis. *Jour. AWWA*, 49:63 (Jan. 1957).
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Discussion

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The information on high-rate filtration contained in the article by Conley and Pitman should be studied by designers and operators of filtration plants throughout the country. The success of filtration at high rates is attributed largely to the use of sand and anthracite together in the filter bed. The authors use a method of turbidity measurement not generally used in water treatment plants. When turbidity as low as 0.005 is reported by their method, the writer wonders if this value can be compared to turbidity values found with instruments in general use.

The authors' article on "Microphotometer Turbidity Analysis" [1] showed the design of the microphotometer they used. The illustration gave the impression that there was a considerable amount of electrical wiring in the instrument, even though all the wiring was contained in a small device. So far, the writer has not used such a microphotometer. The instrument should be considered by water treatment plant operators and tested in a few plant laboratories to see if it gives information of greater value than that obtained with laboratory instruments now in common use.

Turbidity Determinations

While in charge of a water treatment plant laboratory in about 1920, the writer realized that methods of determining turbidity did not give much information on the clarity of the filtered water. Prevailing practice was to look at a sample of water in a 1-gal bottle and compare it with bottled samples containing known turbidities. Usually, if the turbidity was less than about 2, it was recorded as zero. The first step toward a more exact method of testing was the examination of water in a filtered-water basin with a submerged light. When the depth of the

water was 10–20 ft, the submerged light gave a sensitive detection of turbidity. Of the present devices in common use, the submerged light is still the most sensitive one. Unfortunately, many of the submerged lights used throughout the country are so arranged that maximum sensitivity is not obtained.

The next step toward sensitive detection of turbidity was to prepare an instrument with which samples could be tested in the laboratory. The instrument now known as the Baylis turbidimeter [2] was developed in 1923. It accurately detects turbidities between 0.1 and 2.0. Some operators can obtain a turbidity measurement as small as 0.05. If it is essential to treat water so that the turbidity is less than 0.1, a more sensitive device is, of course, advisable. The least turbidity that can be detected with the submerged light is less than 0.1, and seems to be close to 0.01, if the submerged light has been installed to give maximum sensitivity. So small a turbidity measurement is, of course, unnecessary for filtered water in treatment plants. The submerged light, however, does not give a quantitative test.

The submerged-light test is easy to make. The observer turns on a switch at the observation point above the water, looks at the water, and then turns off the switch. The procedure need not take longer than 5–10 sec. Of course, the time used in going to and from the observation point must be considered, for in some plants this point may be a long distance from the laboratory. Utility laboratories are not finding it difficult to produce the desired clarity of water with instruments now used to determine turbidity. If an industrial plant needs water of a better quality than that produced by the water utility, then such a plant should have its own means of making the water clearer. When there is available a microphotometer that can be purchased for a reasonable price and that can make determinations more exact than those of standard turbidimeters, utilities will begin to use it.

Two-Layer Filter Bed

The use of a special filtering medium * overlying sand is not new; such filter beds, however, are not numerous. In the past, most of these beds consisted of a layer of sand deeper than that of the special filtering medium. The possibility of two-layer filter beds was called to the writer's attention in 1935, when experiments with these beds were being conducted at the Chicago Experimental Filtration Plant. In the 1935 annual report of the plant, under the heading "Two-Layer Filter Beds," it is stated that:

> The use of a layer of coarse filtering material of lower specific gravity over a bed of finer filtering material has some possibility of being practical. The specific gravity of the coarser material should be sufficiently less than that of the finer material so that it will remain at the top of the bed. The coarse material serves as a roughing filter and greatly lengthens the filter runs. A 2-in. layer of coarse crushed quartz was placed over a bed of magnetite in one of the glass tube units. The magnetite has a specific gravity considerably greater than that of quartz, and with the sizes used there has been very little mixing of the two materials.
>
> Three beds of sand with 2-in. layers of coarse anthracite coal have been placed in service. One of the small plant filters

* Anthrafilt, a product of Anthracite Equipment Corp., Wilkes-Barre, Pa.

was used for one of the beds, and the other two were glass tube filters. The plant filter has a surface area of 10 sq ft. . . . There was some loss of coal each time the filter was washed, and it appears that the surface washing system cannot be used for beds containing anthracite coal on account of the high loss of coal. On the filter in which the surface wash was not used, there was such an accumulation of mudballs in the top layer of coal and the top of the sand that it would not seem advisable to omit the surface wash. The two-layer bed, with anthracite coal for the top layer, therefore, may not prove practical.

TABLE 8

Cotton Plug Filter Tests for Ash Determinations

Date of Filter Run—*1939–40**		Amount of Ash on Filters—*ppm*			Date of Filter Run—*1939–40**		Amount of Ash on Filters—*ppm*		
Started	Ended	Filter 10 24 in. Special Medium† (0.57 mm‡)	Filter 7 24 in. Sand (0.84 mm‡)	Filter 11 20 in. Sand (0.50 mm‡)	Started	Ended	Filter 10 24 in. Special Medium† (0.57 mm‡)	Filter 7 24 in. Sand (0.84 mm‡)	Filter 11 20 in. Sand (0.50 mm‡)
Aug. 2	Aug. 4	0.127	0.120	0.027	Feb. 19	Feb. 21	0.575	0.467	0.348
7	9	0.106	0.058	0.079	21	24	0.353	0.278	0.386
9	11	0.226	0.021	0.031	26	28	0.549	0.350	0.297
14	16	0.101	0.054	0.037	28	Mar. 1	0.438	0.391	0.474
16	18	0.146	0.034	0.062	Mar. 4	5	0.628	0.595	0.219
21	23	0.155	0.093	0.100	5	8	0.225	0.164	0.087
23	25	0.148	0.059	0.035	11	13	1.315	0.220	0.730
28	30	0.096	0.065	0.054	13	15	1.660	1.100	0.908
30	Sep. 1	0.074	0.029	0.030	14	15	1.434	1.227	0.719
Sep. 6	8	0.105	0.040	0.050	18	19	0.537	1.085	0.831
11	13	0.120	0.113	0.065	19	21	0.028	0.048	0.014
13	15	0.054	0.060	0.019	21	23	0.560	0.863	0.321
18	20	0.140	0.151	0.154	25	27	0.434	0.492	0.371
20	22	0.082	0.040	0.018	27	29	0.318	0.314	0.327
25	27	0.175	0.160	0.174	Apr. 1	Apr. 3	0.590	0.490	0.324
27	29	0.050	0.024	0.021	3	5	0.955	0.908	0.342
Oct. 2	Oct. 4	0.321	0.192	0.162					
4	6	0.043	0.018	0.023				Filter 5‖ 24 in. Sand (0.50 mm‡)	
9	11	0.099	0.074	0.043					
11	13	0.102	0.076	0.023					
16	18	0.073	0.121	0.055					
18	20	0.099	0.098	0.034					
23	25	0.060	0.046	0.124	10	12		0.077	0.548
25	27	0.134	0.037	0.015	15	18		0.151	0.344
30	Nov. 1	0.210	0.029	0.054	18	20		0.332	0.714
Nov. 6	8	0.209	0.207	0.012	22	24		0.398	0.487
8	10	0.259	0.031	0.016	24	27		0.141	0.374
13	15	0.209	0.080	0.018	29	May 1		0.387	0.466
					May 1	3		0.248	0.106
					7	9		0.149	0.188
					9	11		0.082	0.267
					13	15		0.104	0.151
					15	17#		0.040	0.129
					20	24		0.012	0.040
					27	Jun. 1		0.042	0.046
Dec. 11	Dec. 15	0.218	0.090	0.035	Jun. 3	7		0.031	0.056
18	22	0.165	0.056	0.040	10	14		0.048	0.053
26§	29	0.570	0.321	0.158	18	21		0.016	0.057
Jan. 9	Jan. 12	0.324	0.307	0.156	24	28		0.059	0.043
15	19	0.492	0.227	0.202	Jul. 1	Jul. 5		0.042	0.073
23	26	0.245	0.266	0.137	8	12		0.079	0.030
29	31	0.500	0.507	0.219	15	19		0.056	0.026
31	Feb. 2	0.417	0.457	0.133	22	26		0.045	0.055
Feb. 5	9	0.316	0.186	0.315	29	Aug. 2		0.034	0.017

* Aug. 2, 1939–Dec. 26, 1939; Jan. 9, 1940–Aug. 2, 1940.
† Anthrafilt.
‡ Effective size.
§ Start of weak coagulation period.
‖ Filters 7 and 10 taken out of service, Filter 5 substituted.
End of weak coagulation period.

Another reference to two-layer beds appears in the report of the experimental plant for November 1939:

> On several occasions tests have been run on the effect of placing a layer of anthracite coal over the sand in a filter bed, using coal of larger size than the sand. The purpose of these experiments is to use the coarse material of coal at the top for straining out most of the coagulated matter, and the sand for filtering out any material which passes through the layer of coal. The coal has a specific gravity less than that of sand and remains in a layer at the top of the bed after backwashing a filter.
>
> A layer of coal 3 in. thick was placed over the sand in one of the small plant filters. The coal has an effective size close to 1.5 mm, and the sand has an effective size close to 0.5 mm. This arrangement greatly lessens the rate at which the loss of head increases on the filter. Previous experiments indicate that this is a practical means of lengthening the filter runs, and beds of the two materials are being used in several filtration plants throughout the country.

Data on the duration of filter runs before and after the addition of special medium in the filter are found in the records of the Chicago Experimental Filtration Plant. Test runs were conducted during the period Nov. 6, 1939–Dec. 4, 1939. The filter used (Filter 11) was a circular steel tank with 10 sq ft of filtering surface. The bed was rebuilt on Nov. 1, 1939, with about 20 in. of sand, 0.50 mm in effective size. The filter was set to operate at 3.5 gpm/sq ft. The duration of a filter run was the time required for head loss to increase to 8 ft. Without the special filtering medium, filter runs averaged 5.94 hr. Test runs were then made with 4 in. of special medium (the November 1939 report states 3 in.) added to the top of the sand. The effective size of this medium was not recorded; it was probably 16–20 mesh, about 1.5 mm in effective size. With the special medium, the average duration of filter runs was 27.28 hr. A change in the water may have caused some, but not much, of the difference in the duration of the runs.

About 3 months before the tests on duration of filter runs, one of the small plant filters (Filter 10), with 10 sq ft of filtering surface, was filled with about 24 in. of special filtering medium having an effective size of 0.57 mm. Cotton plug filters were in use at the time. The results of cotton plug filter tests for Filters 10, 7, and 11 are shown in Table 8 for the period August 1939–August 1940. The filters were operated at a rate of 3.5 gpm/sq ft. This period included winter months, and there was a long period of weak coagulation, as is shown by the large amount of ash in the cotton plug tests. An ash content of approximately 0.025 ppm is the amount expected in the better operated filtration plants. The effect of acid-treated sodium silicate on the strength of the coagulation is reflected in the 48-hr run started on Mar. 19, 1940, about the middle of the weak coagulation period. The amount of ash on each of the cotton plugs was not large until after the middle of December 1939, and returned to normal in May 1940. The ash content of the cotton plugs increased and decreased on all the filters alike. It was far greater than any amount that could be tolerated during the December–May period.

A major difference in the way these tests were conducted is the thin layer of special medium used in contrast with the thick layer of anthracite used by

Conley and Pitman. The thick layer may be the main reason for the better results.

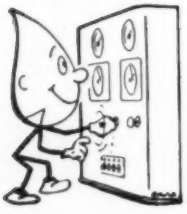
Conclusion

The fact that not many filters with the two-layer bed are in use indicates that there is not much need for them in the average water treatment plant. The results obtained by Conley and Pitman are amazing and should be studied by the water industry, particularly in places where filter runs are short over long periods of time, or where the use of high-rate filtration is being considered.

If water of the highest quality can be produced at filtration rates of 6 gpm/sq ft, fewer filters would be required in new plants, and extension of existing filters would not be needed so early. One important consideration, however, is that the water must be conditioned properly for high-rate filtration.

References

1. Conley, W. R. & Pitman, R. W. Microphotometer Turbidity Analysis. *Jour. AWWA,* 49:63 (Jan. 1957).
2. Baylis, J. R. Sensitive Detection of Suspended Matter and a Proposed Standard of Clarity in Filtered Water. *Jour. AWWA,* 11:824 (Jul. 1924).



Manganese Removal by Oxidation With Potassium Permanganate

Reginald B. Adams

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METHODS available for the removal of manganese from water supplies can normally be classified as high pH, oxidation, or a combination of the two. The high-pH method has been recognized as one of the simplest, yet it has been one of the most misused. High-pH methods have been selected quite often without concern for side problems, such as calcium carbonate stabilization, poor coagulation, and color, taste, and odor removal. Oxidation of manganese with chlorine has been used extensively in various pH ranges. In many instances, the speed of reaction with chlorine has been too slow, or the design of the plant has been such that the oxidation of manganese has not been complete in the flocculation stage, with the resultant carryover of the fine manganese precipitate to the filters.

Potassium permanganate has been used successfully for the regeneration of manganese ion exchanger. This process utilizes permanganate as a powerful, fast-acting oxidizing reagent and one that does the job usually in the neutral or slightly alkaline pH range. Potassium permanganate can be used basically in the same manner to oxidize the manganese in a conventional water treatment plant. The oxidation is fast in a slightly alkaline pH range.

Theoretical Considerations

Latimer [1] gave the following oxidation states of manganese:

1. Manganese forms compounds of the manganous ion, Mn^{++}, and the manganic ion, Mn^{+++}. Their corresponding oxides are basic.

2. The Mn^{4+} oxide, MnO_2, is amphoteric—that is, it is capable of reacting as either electropositive or electronegative, and as either an acid or a base. It is comparatively inert toward hydrogen ions and hydroxide ions.

3. An ion of the Mn^{5+} state—hypomanganate, probably MnO_3^-—has been recently observed, but it is highly unstable in both acidic and basic solutions.

4. The Mn^{6+} oxidation state is represented by the manganate ion, MnO_4^{--}, which is stable only in alkaline solution.

5. The Mn^{7+} oxidation state is represented by permanganate, MnO_4^-.

Kolthoff and Belcher [2] have listed the general equations for permanganate oxidations in acid and alkaline solutions. In acid solution, the reaction is

$$MnO_4^- + 8H^+ + 5e \rightleftharpoons Mn^{++} + 4H_2O.$$

The oxidation potential of a permanganate-manganous system is influenced greatly by the hydrogen ion concentration in acid solution. In weakly acid,

neutral, or alkaline solution, the reaction is

$$MnO_4^- + 4H^+ + 3e \rightleftharpoons MnO_2 + 2H_2O.$$

In this reaction, the hydrogen ion concentration has less effect on the oxidation potential of a permanganate-manganous system than it has in a stronger acid solution.

The reaction of permanganate with manganous ion in neutral solution (Volhard's reaction) is

$$3Mn^{++} + 2MnO_4^- + 2H_2O \rightarrow 5MnO_2 + 4H^+.$$

Hydrated MnO_2 has properties of an acid and adsorbs manganous ions, preventing their complete oxidation. The difficulty can be avoided, as Volhard demonstrated, by introducing a sufficient amount of calcium, magnesium, barium, or zinc salt. These cations are adsorbed preferentially, leaving manganous ions in the solution free to react. The theoretical amount of $KMnO_4$ required to oxidize Mn^{++} is: $3Mn^{++}:2KMnO_4 = 165:316$, or 1.0 part Mn^{++} requires 1.9 parts $KMnO_4$.

The reaction of potassium permanganate with Mn^{++} in neutral or alkaline solution oxidizes the Mn^{++} and reduces the MnO_4^- to hydrated MnO_2. The pH of probable partial precipitation is less than pH 5.0; the pH of complete precipitation is about pH 8.0.[3]

Potassium permanganate will oxidize a great variety of substances in the acid, neutral, and alkaline ranges.[2] Some of these substances include sulfide, sulfite, thiosulfate, cyanide, ferrous iron, manganous manganese, chromic chromium, and numerous organic compounds such as organic acids, alcohols, aldehydes, ketones, phenols, and various nitrogen compounds. Generally speaking, the oxidation of an organic compound with permanganate proceeds more rapidly in an alkaline solution than in an acid solution.[2]

Permanganate does not oxidize ammonia to nitrogen or nitrogen oxides quantitatively. In an alkaline medium, there is some formation of nitrite. In neutral or alkaline solution, permanganate does not react with nitrite.[2]

Manganese in Raw Water

Manganese has been a problem at the Wilkinsburg, Pa., treatment plant for many years. The source of supply originally and prior to the construction of the treatment plant was the Allegheny River at Nadine, Pa. The raw-water intake is 8.9 mi above the confluence of the Allegheny and Monongahela rivers at the point in Pittsburgh. The main Allegheny River north of the confluence with the Kiskiminetas River, 21.3 mi upstream from the Wilkinsburg intake, has long been known to contain small amounts of manganese. The Kiskiminetas, a major tributary of the Allegheny, has been known to contain large amounts of manganese. The Kiskiminetas watershed includes an area of western Pennsylvania that has been the site of extensive bituminous coal–mining operations. The Loyalhanna River, a major tributary of the Kiskiminetas, has a normal manganese content of 0.5–8.0 ppm, depending upon flow conditions. The source of the manganese is coal mine drainage.[4–6] The Conemaugh River, a second major tributary of the Kiskiminetas, has a normal manganese content of about two times that of the Loyalhanna; yet the acid streams flowing into the Conemaugh as a result of coal mine drainage carry approximately the same manganese content as those flowing into the Loyalhanna. The noticeably large difference in the man-

ganese content of Conemaugh water as compared to Loyalhanna water is attributed to the operation of a large steel mill on the headwaters of the Conemaugh.

The soluble manganese content of Allegheny River water at the Wilkinsburg intake has been 0.5–4.0 ppm, with a normal yearly average of 1.20 ppm.

A secondary source of supply was utilized at Wilkinsburg during the period 1896–1902, when filter cribs made of sand and gravel were constructed in the bed of the Allegheny River and connected to the raw-water intake. A mixture of filtered river water and ground water became the main source of raw-water supply until the treatment plant began to operate in 1910. Gradually, the percentage of raw river water increased as a result of the demand on the system, until it became difficult to detect the presence of ground water. The ground water had a manganese content of approximately 1.50 ppm, and undoubtedly it increased the manganese content of raw water in the early days of plant operation, and to a lesser extent in recent years.

Problems With Manganese

In 1930, the treatment at the Wilkinsburg plant consisted of coagulation with alum and lime at a pH of 6.8–7.0, sedimentation, filtration through sand, and postchlorination. Filter runs averaged 24 hr. The fine sand grains were coated with manganese dioxide deposits; the coarse sand grains were stained dark brown with manganese dioxide, and the gravel beds were heavily coated and cemented with it. The distribution mains, service lines, and meters carried heavy deposits of manganese dioxide. Manganese removal in the plant averaged 18–20 per cent. Consumer complaints about the water, with regard to the laundering of clothes and the staining of porcelain fixtures, were numerous.

Treatments for Manganese Removal

At Wilkinsburg, prechlorination in the marginal range resulted in very little, if any, improvement in manganese removal. Various pH ranges were tried, with manganese removal improving as the pH was increased. Complete manganese removal was obtained when the pH was such that 2.0 ppm of causticity resulted. To bring this about, the pH range had to be approximately 9.4–9.6. The water was strongly buffered at this range, to the extent that control of manganese removal by pH was not nearly as sensitive as control by the causticity test. Unfortunately, high-pH treatment without recarbonation places a heavy load on the filters and soon causes the sand grains to become coated until they are no longer effective for removing fine precipitates.[7] There were no facilities for recarbonation at the plant. Attempts were made to stabilize calcium carbonate just ahead of the filters by using polyphosphate, but these attempts were unsuccessful.

Laboratory studies involving the use of free residual chlorination were made at the Wilkinsburg plant. It was concluded that oxidation of manganese with chlorine at the breakpoint (1.5-hr contact) and at pH 8.0 was a slow reaction and that considerable contact time should be provided before mixing and coagulation. The latter observation was more or less confirmed with plant scale runs at Natrona, Pa. The plant at Natrona used Allegheny River water as the source of supply. Free residual chlorination at pH 8.0 did not

oxidize the manganese quickly enough for the manganese precipitate to be adsorbed in the coagulation process. As a result, the filters had to remove most of the manganese as a fine precipitate.

Oxidation of manganese with chlorine dioxide in conjunction with prechlorination in the marginal range was tried on a plant scale for several months during 1947. Laboratory experiments had indicated that chlorine dioxide would oxidize manganese in the pH range 8.0–8.5, but in plant practice a range of 8.5–9.0 had to be used. Again calcium carbonate deposition was excessive and the treatment was expensive. A minimum of 2 lb/mil gal sodium chlorite was required; at times the dosage was as high as 5 lb/mil gal. One thing in favor of chlorine dioxide should be mentioned. The speed of reaction was such that 65 per cent of the manganese was removed in the sedimentation basins.

Advantages of Permanganate

Rapid and complete removal of soluble manganese in the distribution mains led to the suspicion that rapid oxidation was taking place soon after the water left the treatment plant. Samples of manganese deposits were taken by flushing hydrants of the distribution system. These deposits of manganese, when added to and stirred in samples of filtered water containing soluble manganese, completely eliminated soluble manganese. Similar deposits added to samples of prechlorinated raw water that had been adjusted to pH 7.2–7.4, completely eliminated soluble manganese. The manganese deposits from the distribution mains were assumed to be manganese dioxide. Samples of chemically pure manganese dioxide, however, did not produce the same effect as the material taken from the mains; in fact, these samples gave no reaction. The only conclusion to be reached from these results was that there should be found an oxidation state of manganese greater than Mn^{4+}. An attempt to find a higher valence of manganese led to the common laboratory reagent potassium permanganate. A very rough test indicated that potassium permanganate would remove soluble manganese from the raw water in much the same manner as did the deposits from the mains.

Jar tests demonstrated that potassium permanganate would rapidly oxidize soluble manganese, generally in the pH range 7.5–8.0, and that the resulting precipitated manganese could be coagulated. In 1941, a plant trial run was indicated, but when the purchasing department tried to get potassium permanganate, they were informed that it was an import from Germany and that shipments had been stopped because of the international situation. Laboratory work continued until potassium permanganate became available in 1948.

In the meantime, an article was published relative to the use of potassium permanganate for the removal of manganese from a water supply used at a paper mill.[8] Permanganate was used during the period 1933–41; then it became unavailable because of the war. The ratio of permanganate to manganese was slightly greater than 2:1. It is assumed that prechlorination was not used with the permanganate treatment.

Studies made at Wilkinsburg, by Chamberlin, Kinney, and Adams in 1948, demonstrated that superchlo-

rination, with as much as 8–10 ppm chlorine beyond the breakpoint (1.5-hr contact), was necessary in order to get the speed of reaction at pH 8.0 comparable to that of permanganate. Furthermore, it was concluded that the most logical and economical method of oxidizing manganese was with a combination of chlorine and permanganate.

Plant Scale Treatment

Potassium permanganate was first tried on a full plant scale in 1948. It has been used constantly since that time. There was very little information available on the use of permanganate for manganese removal in water treatment, aside from its use in regenerating manganese ion exchanger filters. Day-to-day observations, problems, and experimental and plant scale data are discussed below.

Method of Feed

Small capacity plants, 1.0 mgd or less, find it advisable to feed permanganate as a 2 per cent solution, with a proportioning pump. Larger plants will more likely prefer dry-feed methods. The Wilkinsburg plant has used dry-feed equipment to feed permanganate at rates between 6 mgd and 25 mgd. Ordinary technical-grade potassium permanganate is too hygroscopic to feed dry, unless a drier is added.

Ground alum, 10 per cent by weight, was admixed with permanganate as a drying agent during normal weather conditions. The amount of alum was increased to 15 per cent for conditions of extremely high humidity. Recently, however, a free-flowing potassium permanganate has been developed.* This new chemical has been used on an experimental basis for a period of 6 months and has been found to feed satisfactorily without the addition of a drying agent at the plant. It is advisable to work the material through a $\frac{1}{8}$-in. screen when charging storage hoppers.

Technical-grade potassium permanganate is packaged in 25-lb kegs, 110-lb kegs, and 600-lb drums. Free-flowing potassium permanganate can be handled pneumatically as long as organic matter is not introduced.

Order of Chemical Additions

Chemical equations involving the oxidation of Mn^{++} with permanganate in the pH range 7.2–8.3 have been suggested earlier. In actual practice, chlorine is added to at least the marginal range of 0.3–1.0 ppm total available chlorine. This oxidizes the ferrous iron and partially satisfies the chlorine demand of the water, which would otherwise require more permanganate. Chlorine can be used in the marginal, intermediate, or free residual range. The balance of the oxidation potential is supplied by permanganate. The permanganate oxidation when used with chlorine is, therefore, partially selective to the manganous manganese, and the end product is hydrated manganese dioxide. The latter is of fine particle size at the pH used and, when present in amounts exceeding 1.50 ppm as Mn, creates a real coagulation problem. It follows that the higher the concentration of manganese the more difficult the coagulation problem. It is believed that practically all the Mn^{++} manganese should be oxidized so as to take advantage of heavy flocculation and sludge contact. Oxidation taking place beyond the floc zone puts a manganese

* The material was developed in 1958, by the Carus Chemical Co., La Salle, Ill.

TABLE 1

Speed of Oxidation by Permanganate at Various pH Levels

pH	$KMnO_4$ Required* *ppm*	Reaction Time† *min*
5.0	1.8	<5
6.0	1.6	<5
7.2	1.4	<5
8.0	1.4	<5
9.0	1.0	<5

* The experimental dosage of permanganate required is somewhat higher than that used in practice.
† Because of the time necessary for the addition and adjustment of chemicals, and because of the short mixing period and filtering period, reaction times of less than 5 min could not be estimated.

load on the filters. The manganese precipitate is fine, and the penetration can be quite deep into the filters. The major portion of the manganese should be removed in the sedimentation basins.

The order of adding chemicals was somewhat different during the early development stages from the order at the present time. At first, the order was chlorine (5–10-min contact), alum, lime, and permanganate. Later, the use of polyelectrolytes changed the order to chlorine, lime, permanganate, alum, and polyelectrolyte. The order of chemical additions is very important in laboratory studies because of the time lag between additions, and it is well to adhere to the same order in plant practice. It is also well to remember that the permanganate should not be added to an acid solution, because the amount of permanganate required will then have to be increased. The permanganate will oxidize manganese in acid solution, but it will not necessarily be as selective to manganese, which accounts for the greater quantity of permanganate needed. The explanation of this is evidently related to the fact that permanganate carries a higher potential in very acid solution than in neutral or alkaline solution; otherwise the change in order of adding chemicals is primarily done to alleviate the coagulation problem.

Speed of Reaction

One of the main advantages of the permanganate treatment for the oxidation of Mn^{++} is the speed of reaction. The manganese can be oxidized and precipitated during the usual mixing and flocculation period. An attempt was made to check the reaction time at various pH levels, but the time was so short that only an estimation could be made. A synthetic solution was prepared containing 1.0 ppm soluble manganese and 0.5 ppm total residual chlorine. Aliquot portions were treated with permanganate at various pH levels and the time for oxidation measured. Results are summarized in Table 1.

pH Range

The pH range for oxidation at the Wilkinsburg plant has been 7.4–8.3. The oxidation reaction itself is not limited to the above pH range. The lower limit, pH 7.4, has been established by experience as the minimum pH at which 0.50 ppm manganese can be speedily and completely oxidized

TABLE 2

Relationship Between Manganese Concentration and pH Range Relative to Permanganate Treatment

Soluble Mn Concentration in Raw Water *ppm*	pH Range for $KMnO_4$ Treatment
<0.5	7.2–7.4
0.5–1.0	7.4–7.6
1.0–1.5	7.6–7.8
1.5–2.0	7.8–8.0
2.0–4.0	8.0–8.3

and the manganese colloid broken and properly conditioned for filtering. The slightly extended pH range of 7.2–8.3 will apply in many instances when low concentrations of manganese are encountered. Table 2 illustrates the relationship between pH range and manganese concentration relative to the permanganate treatment.

Coagulants and Coagulant Aids

Alum and ferric sulfate. The selection of the best coagulant has been one of the most difficult phases of the permanganate treatment. Serious consideration must be given the fine colloidal manganese precipitate formed within seconds after the addition of permanganate when the precipitate is to be coagulated in the pH range 7.4–8.3. Alum was first tried as the coagulant. It worked well, except for a problem involving soluble aluminum-silica complex, a problem that became evident particularly during periods of low water temperature. A ferric sulfate coagulant was tried on a plant scale for several months. Contrary to some references in the literature, ferric sulfate was not as effective as alum for the colloids of manganese in the pH zone 7.4–8.3. Incidentally, it was also concluded that colloids of iron resulting from the oxidation of ferrous iron in low-temperature raw water with chlorine, were more effectively coagulated with alum than with ferric sulfate in the pH zone 7.4–8.3.

Activated silica. A search for more efficient methods for the coagulation of manganese colloids led to experimental work with activated silica. Laboratory results were most encouraging.[9] The sodium bicarbonate method of activation was used. Plant scale operation followed closely the data obtained in the laboratory.

Activated silica does not act directly on the manganese colloids, as far as can be determined, but it does aid the coagulant alum in breaking up the colloid. It is quite apparent that the manganese colloid does not carry a strong positive charge in the pH range 7.4–8.3, owing to the lack of reaction between the activated silica and manganese dioxide. It is suggested that the manganese colloid is broken by adsorption of calcium and magnesium ions at pH range 7.4–8.3, and that the aluminum-silica floc is more efficient than straight alum floc in adsorbing the finely precipitated manganese dioxide. Where time of mixing, flocculation, settling, and high coagulant dose are not a matter of concern, excellent adsorption of the manganese colloid can be accomplished with a good alum

TABLE 3

Effects of Polyelectrolytes Used With Permanganate Treatment

$KMnO_4$ Treatment	Duration of Filter Runs *hr*	Wash Water Used %	Mn Removed by Basins %	Fe Removed by Basins %	Period of Time Used *yr*
Without polyelectrolytes	60	2	71.3	89.4	4
With polyelectrolyte*	67	1.9	74.2	91.4	3
With activated silica	73	1.8	74.7	91.6	4

* Hagan's coagulant aid HCA-18.

TABLE 4

Average Quantities of Chemicals Used With Permanganate Treatment for Manganese Removal at Wilkinsburg

Chemical	Amount Used *lb/mil gal*
Chlorine	19
Lime	186
Potassium permanganate	10
Alum	257
Coagulant aid	25

floc and without the use of a polyelectrolyte. Experience has shown, however, that the use of a polyelectrolyte involves no additional cost, because of the corresponding reduction in coagulant needed, and because of the greater efficiency in the coagulation process. The filtrability of the polyelectrolyte-treated water is readily demonstrated in longer filter runs (Table 3). The aluminum-silica complex problem mentioned previously is not to be confused with the use of activated silica. Actually, the activated-silica process helped to reduce this problem, but not to a satisfactory degree. The real answer to the aluminum-silica complex problem at Wilkinsburg will be a two-step treatment whereby the pH will be lowered in the second step after the oxidation of manganese for complete aluminum removal.

Coagulant aid. Activated silica was used on a plant scale for 4 years, until a packaged polyelectrolyte * became available. This has been used for approximately 3 years, primarily because of the ease of handling and feeding. No activation is required. The material is dry fed. An excess of water is added to make a slurry, and from this point on, the polyelectrolyte is used in the same manner as activated silica, even to the same average dose of 3.0 ppm. Table 3 shows the effect of polyelectrolytes on plant operation.

* Hagan's coagulant aid HCA-18, made by Hagan Corp., Pittsburgh, Pa.

Chemical Dosage

Table 4 lists the average quantities of chemicals used with the permanganate treatment for manganese removal at Wilkinsburg. These quantities are not entirely chargeable to the manganese removal treatment. Sterilization, clarification, iron removal, and odor reduction are accomplished at the same time.

The exact ratio of chlorine to permanganate will depend on the treatment problems and on the preference of the operator directing the treatment. For example, the work at Wilkinsburg has been done almost entirely with prechlorination in the marginal range (0.3–1.0 ppm total available chlorine) plus 1.0–1.2 ppm permanganate for every part of soluble manganese in the raw water. Table 5 shows the amount of permanganate required to oxidize 1.0 ppm manganese at various levels of prechlorination of one type of water at pH 8.0.

TABLE 5

*Amounts of Permanganate Needed to Oxidize 1 ppm Manganese at Various Prechlorination Levels**

Chlorine Residual by Test—*ppm*		$KMnO_4$ *ppm*
Free	Combined	
0.00	0.00	2.2
0.00	0.50	1.2
0.40	1.70	0.9
4.0	0.90	0.6
12.0	0.00	0.0

* The prechlorination-permanganate combinations are set up on the basis of a fast reaction—that is, the chlorine contact time is approximately 10 min and the mixing time with permanganate is 10 min.

The breakpoint on the above water is usually in the range 5–10 ppm chlorine for a 1.5-hr contact period, and 12–20 ppm chlorine for a 10-min contact. The only way to bring about a fast oxidation of manganese with chlorine alone at pH 8.0 is to add enough chlorine to burn up instantaneously the ammonia or other chlorine-consuming compound that is taking precedence over the manganese in the order of reaction.

Manganese Removal

Table 6 shows the manganese content of raw, settled, and filtered waters at the Wilkinsburg treatment plant for the period 1948–1958. Under the operating conditions stated previously, the permanganate feed rate has been set so that the manganese in filtered water will not exceed 0.10 ppm. The ratio of permanganate feed to the soluble manganese in the raw water is established periodically by means of jar tests.

Filter Operation

One of the main advantages of the permanganate treatment for manganese removal is that the reaction is carried out in the pH range 7.2–8.3, depending on the manganese concentration in the raw water. There is no calcium carbonate to deposit on the filter media; and as long as enough permanganate is added to oxidize all the soluble manganese before the water reaches the filters, there will be no manganese deposit on the filter media or supporting gravel bed. Even low concentrations of soluble manganese, in the order of 0.10–0.15 ppm, resulting from undertreatment with permanganate for short periods, present no serious problem with the filters, as far as deposition of manganese is concerned. A sand filter rebuilt in 1950 is stained brown and black all the way to the gravel. There is no significant change in effective size of the sand, which is still sharp. The gravel bed is clean. New filter sand in other filters, after 1 year of use, is only slightly stained.

Under perfect conditions of operation, there have been filter runs as long as 150 hr when a special filtering medium * was used. The average filter run with this filtering medium, having an effective size of 0.70–0.80 mm, is approximately 72 hr; the average filter run with sand having an effective size of 0.55 mm is approximately 48 hr. The governing factor of the run with the special medium is the penetration of fine, precipitated man-

TABLE 6

Manganese Removal Data for Wilkinsburg Treatment Plant, 1948–1958

Year	Average Amount of Manganese—*ppm*		
	Raw Water	Settled Water	Filtered Water
1948	1.03	0.37	0.07
1949	1.09	0.35	0.02
1950	0.95	0.25	0.03
1951	1.28	0.29	0.04
1952	1.22	0.28	0.04
1953	1.23	0.31	0.05
1954	1.15	0.28	0.06
1955	1.33	0.37	0.09
1956	0.98	0.28	0.05
1957	1.51	0.35	0.06
1958	1.00	0.29	0.05
Avg	1.16	0.31	0.05

* Anthrafilt, a product of Anthracite Filter Corp., Wilkes-Barre, Pa.

ganese; the governing factor of the sand run is loss of head.

Summary

The permanganate method of manganese removal is based on the oxidation of Mn^{++} to MnO_2 at pH 7.2–8.3. Chlorination before the permanganate treatment appreciably reduces the amount of permanganate required. The reaction is rapid enough to allow coagulation of the fine, precipitated manganese during the normal coagulation process. Polyelectrolytes have aided the coagulant alum in producing a high-quality filtrable water.

Acknowledgments

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Evaluation of Membrane Filter Technique for Appraising Ohio River Water Quality

—*Harold W. Streeter and David A. Robertson Jr.*—

A report originally prepared for the Ohio River Valley Water Sanitation Commission (Orsanco), by Harold W. Streeter, Staff Consultant, and David A. Robertson Jr., San. Engr.-Hydrologist, both of Orsanco, Cincinnati, Ohio. According to Orsanco's executive director and chief engineer, Edward J. Cleary, the findings of this study resulted in the adoption by Orsanco, in 1958, of the membrane filter technique as an optional alternative to the MPN procedure for bacteriologic analysis of river water. Several of the Ohio River Compact signatory states have approved the use of the membrane filter technique by water plant laboratories that have demonstrated the ability to utilize it.

THE membrane filter (MF) technique for the bacteriologic examination of water, formally introduced into the United States in 1947, has been used in Europe for many years. Both Germany and Russia relied on this method during World War II, when laboratory facilities for other methods were not available. Since 1947, much study in the United States has been devoted to the membrane filter and its uses, particularly to its applicability for the enumeration of coliform bacteria in water. Studies have been made primarily to improve the MF test and to evaluate its advantages and limitations. With regard to the latter, special interest has been shown in the comparisons between the MF technique and the standard dilution test, the latter of which is expressed in terms of the most probable number (MPN).

Because the eight states—Illinois, Indiana, Ohio, Virginia, New York, Pennsylvania, West Virginia, and Kentucky—signatory to the Ohio River Valley Water Sanitation Compact have for some years been sponsoring a river quality monitoring program, it seemed appropriate that they should become intimately informed on the applicability of the MF technique. Thus, the Ohio River Valley Water Sanitation Commission (Orsanco) staff was authorized in 1956 to undertake an evaluation.

The study was carried out as a collaborative project with the Orsanco Water Users Committee. Members of the committee include the managers of municipal water treatment plants on the Ohio River and some of its tributaries. They serve the commission primarily in the collection of data on river quality; one of the routine tests is the measurement of coliform bacteria concentrations. Eight stations were selected initially for the study, in order to secure data under varying conditions. Two of the stations, however, had to withdraw before all the tests were completed. All persons, except one, who participated in the study had no previous experience with the MF technique.

To implement the investigation, the commission appropriated funds for the purchase of equipment and supplies. In addition, the commission defrayed the expenses of the participants who were assembled in Cincinnati for orientation at the Taft Sanitary Engineering Center of USPHS. Here the participants had the opportunity, under the direction of Harry Kramer, chief of the training section, to observe procedures and discuss details with Paul Kabler and Harold Clark, both of whom are authorities on the conduct of the MF test.

In November 1956, the eight laboratories began making MF determinations along with the MPN test. Normal procedure was to compare the two tests on samples of river water collected and examined on 5 days each week. The plant laboratories in which the tests were made are shown in Table 1.

From time to time during the test period, which lasted for 20 months, briefing sessions were held among the participants. These sessions provided the opportunity to exchange experiences, a practice that proved helpful in improving the skill of each participant. Participants in neighboring laboratories also visited each other occasionally to cross-check their results on the same samples. To guide the participants further in attaining uniformity of methods and procedures, the group commissioned a subcommittee—consisting of R. B. Adams, of Wilkinsburg, Pa., and Dan Enright, of Cincinnati—to draft a memorandum for carrying out the comparative tests (Appendix A). As another aid to facilitate the interpretation of MF results, Adams contributed a valuable paper on this subject, based on an analysis of the results from previous MF work done at his laboratory and those reported in an article by Thomas and Woodward.[1] The article deals specifically with planning the MF tests to give the optimum range of countable coliform colonies on the filter pads, and discusses methods for selecting the filter counts to be used in arriving at the final result from an individual test. The staff recommendation, based on Adams' analysis, is given in Appendix B.

Those members of the Water Users Committee who undertook the study as a voluntary activity contributed laboratory facilities and personnel free of charge. With this generous cooperation, the commission found it possible to complete the investigation at the nominal cost of $4,500.

TABLE 1

Location of Plant Laboratories Used in Orsanco Evaluation of MF Tests

Plant Laboratory	River	Distance From Pittsburgh *mi*
Wilkinsburg-Penn Joint Water Authority	Allegheny	8.9
South Pittsburgh Water Co.	Monongahela	4.5
Weirton Water Plant	Ohio	61.7
Wheeling Water Plant	Ohio	86.8
Portsmouth Water Plant	Ohio	350.7
Cincinnati Water Plant	Ohio	462.8
Louisville Water Co.	Ohio	600.6
Evansville Water Plant	Ohio	791.5

Experiences and Results

The investigation may be divided into three periods: Nov. 1, 1956–Apr. 30, 1957; May 1, 1957–Nov. 30, 1957; and Jan. 1, 1958–Jul. 31, 1958. The first period was one of adjustment, during which the several laboratories endeavored to bring their results into harmony by perfection of method.

This period was marked by frequent discussions among the participants and by experimental work with different procedures. The second period was one of marked improvement in results, but some questions as to the best selective culture medium for routine MF work remained unanswered. The third period was one of comparative trials of the one-step Hajna-Damon (HD) medium, which had been used up to that time, and a two-step HD medium involving preliminary enrichment; the objective was to determine which of the two media would give higher bacteria counts. During June and July 1958, a third medium, designated as the modified one-step MF Endo, was compared to the two-step HD medium, with results that appeared promising.

Agreement of Data

A summary of the comparative results for the first and second periods of the study is shown in Table 2. When the combined data from all the laboratories for the first 6-month period are considered, the agreement between MF and MPN counts was 64 per cent when based on individual daily results; 61 per cent, based on monthly averages; and 70 per cent, based on monthly medians. During the next 7-month period, the agreement increased to 81 per cent for daily results and monthly averages, and to 90 per cent for monthly medians. These results reveal the striking improvement that took place between the two periods of testing.

Results were "in agreement" if the MF count was not less than 0.3 nor more than 3.0 times the corresponding MPN count, a range that defines the variability (95 per cent confidence limits) of the MPN counting procedure where five tubes are planted in each dilution. This is a wide range, but it takes into account the normal error involved in the MPN enumeration.

Another test of agreement was a practical one: Were both the MF and MPN counts greater or less than 5,000 per 100 ml? This value represents the Orsanco bacterial-quality objective for coliform density in raw waters taken from the Ohio River for treatment. Lack of agreement in this respect would mean that a raw water would meet or fail to meet this objective according to whether reliance was placed on the monthly average MF count or on the standard MPN count. During the first 6-month period, the agreement in this respect (as shown in Table 2) was only 57 per cent; but during the next 7-month period, it increased to 95 per cent.

Among the several laboratories, excellent agreement for individual daily results was shown at Wilkinsburg, Weirton, Portsmouth, and Louisville (after a period of adjustment in method). Good agreement was found at Cincinnati, South Pittsburgh (in the latter period), and Evansville. At the Wheeling laboratory, where a range of 10–71 per cent was observed from month to month, the agreement was only fair. Thus, with one exception, the general agreement was good to excellent. The overall agreement from month to month for all the laboratories combined varied from 58 per cent to 89 per cent for daily results, with a preponderance of percentages in the high 70's and 80's.

These results compare very favorably with those observed elsewhere in similar comparative tests, and serve to demonstrate what can be accomplished by coordination of effort and attention

to detail. They also reveal that a period of adjustment and experience with the MF technique is necessary before consistently good results are obtainable. This is true of any method involving bacteriologic procedures.

A noteworthy feature of the results was the better agreement shown between the MF and MPN counts when expressed as monthly medians than when expressed as monthly averages. For example, the overall agreement between monthly medians was 80 per cent, as compared with 71 per cent for the monthly averages. It is difficult to explain this difference, but it may be that the coliform counts, like those of total bacteria, tend to follow a normal logarithmic distribution, and the median count is a reflection of this tendency. It is a matter of general observation that the median count tends to be closer to the logarithmic mean than to the arithmetic mean. The difference was not serious in the example cited; it served only to impose a more severe test of agreement between the monthly average MF and MPN counts.

Comparison of Different Media

The third phase of the investigation, from January through July 1958, was concerned with comparative tests of three different media for confirming the presence of coliform organisms on the membrane filters. Throughout the period of November 1956–November 1957, a one-step medium proposed and used by Hajna and Damon,[2] designated as "one-step HD," had been used in the tests by all the participating laboratories. The results obtained

TABLE 2

Summary of Comparative MF and MPN Results

Plant Laboratory	Period	Percentage of MF Counts Within Range of 0.3–3.0 of MPN's*			Months of Agreement†
		Individual Results	Monthly Average	Monthly Median	%
Wilkinsburg‡	Nov. 1956–Nov. 1957	83	100	100	100
S. Pittsburgh	Jan. 1957–Aug. 1957	51	50	50	100
	May 1957–Aug. 1957	67	75	100	100
Weirton	Nov. 1956–Nov. 1957	84	85	85	54
	May 1957–Nov. 1957	99	100	100	86
Wheeling‡	Nov. 1956–Nov. 1957	48	18	54	54
Portsmouth‡	Nov. 1956–Nov. 1957	100	100	100	100
Cincinnati	Nov. 1956–Nov. 1957	64	58	83	75
	Jun. 1957–Nov. 1957	59	50	83	83
Louisville	Jan. 1957–Nov. 1957	62	70	60	70
	Jun. 1957–Nov. 1957	96	100	100	100
Evansville	Nov. 1956–Apr. 1957	76	67	100	33
Weighted Avg	Nov. 1956–Nov. 1957	73	71	80	75
	Nov. 1956–Apr. 1957	64	61	70	57
	May 1957–Nov. 1957	81	81	90	95

* Not less than 0.3 nor more than 3.0 times the corresponding MPN count.
† Months when both MF and MPN average counts were greater or less than 5,000 per 100 ml.
‡ No material change in agreement after first 6 months of tests.

with this medium showed a greatly improved agreement with those of the parallel tests by the MPN method after the first 6 months of the period, as indicated by Table 2. But because these results consistently had the tendency to be smaller in magnitude than the corresponding MPN's, the Water Users Committee believed it desirable to find a medium that would cause the results to agree more closely.

In response to this further interest expressed by the participants, the commission authorized an extension of the study to a series of tests comparing the one-step HD medium with other media. For this purpose, use was made first of a two-step medium originally developed at the Taft Center and described in *Standard Methods*.[3] As it was also used by Hajna and Damon in their study, it has been designated as "two-step HD." Methods followed were those in Appendix C, which were prepared by Adams and Shroyer.

The comparative tests of these two selective media were continued for 5 months, from January through May 1958. A third medium, MF Endo,* [4] appeared so promising that it was tested against the two-step HD medium for 2 months in June and July. This medium, which involves only a one-step procedure, will be designated as "one-step MF," to distinguish it from the one-step HD medium.

The results of this entire series of comparative tests are summarized in Table 3, in terms of monthly averages. For all the tests, the two-step medium averaged 20 per cent higher counts than the one-step HD medium. If one abnormally high ratio based on very low counts is eliminated, the one-step MF medium averaged 16 per cent higher counts than the two-step HD medium. From this limited number of tests, the one-step MF medium showed higher counts than the two-step HD medium in ten out of twelve monthly averages; only one average count was actually lower.

If the magnitude of coliform recovery had been taken as the criterion of satisfactory performance, the two-step HD medium would have shown a clear advantage over the one-step HD medium, and the one-step MF medium almost as much advantage over the two-step HD medium. The consensus among the participants favors the routine use of the one-step MF medium on the basis of the comparative performance of the three tested media and the greater simplicity of the one-step procedure.

According to an investigation reported by Adams, the addition of ethanol to the one-step MF medium increased the clear-cut sheen in the typical coliform colonies. Further experience will undoubtedly result in the improvement of this medium and, possibly, in the development of other confirmatory media giving still better performance. The present series of comparative tests has shown that acceptable results can be obtained by careful technique and experience in conducting the MF test.

Precision and Accuracy

The high degree of reproducibility of the results from the MF tests is shown in Table 3 by the close correlation of corresponding average counts obtained from the tests with the three different media. Despite the average trend of the results, they had a tend-

* Developed by the Millipore Filter Corp., Watertown, Mass., and available through Baltimore Biological Laboratory, Baltimore, Md.; Difco Laboratories, Detroit, Mich.; and Albimi Labs., Brooklyn, N.Y.

TABLE 3

Comparative Results of MF Media Investigation

Laboratory and Month—*1958*	Monthly Average Coliform Count per 100 ml			Ratio of Counts	
	1-Step HD	2-Step HD	1-Step MF	2-Step HD / 1-Step HD	1-Step MF / 2-Step HD
Wilkinsburg					
Jan.	1,600	2,500		1.56	
Feb.	3,500	3,500		1.00	
Mar.	960	1,200		1.25	
Apr.	1,600	1,900		1.19	
May	950	1,100		1.16	
Jun.		4,900	5,700		1.16
Jul.		8,500	7,800		0.92
Weirton					
Feb.	1,400	1,800		1.36	
Mar.	2,000	2,300		1.15	
Apr.	2,500	3,100		1.24	
May	1,900	2,300		1.21	
Jun.		1,400	1,500		1.07
Jul.		580	700		1.21
Wheeling					
Jan.	970	970		1.00	
Feb.	470	420		1.00	
Mar.	1,100	1,200		1.09	
Apr.	2,000	2,600		1.30	
May	3,300	3,400		1.03	
Jun.		620	700		1.13
Jul.		58	250		4.31
Portsmouth					
Jan.	3,700	6,200		1.68	
Feb.	2,000	2,900		1.45	
Mar.	4,300	5,400		1.23	
Apr.	5,600	5,900		1.05	
May	6,600	7,800		1.18	
Jun.		6,900	7,300		1.06
Jul.		13,000	13,000		1.00
Cincinnati					
Feb.	3,900	5,400		1.36	
Mar.	4,300	5,400		1.26	
Apr.	5,200	6,200		1.19	
May	5,500	7,500		1.36	
Jun		7,900	9,900		1.25
Jul.		21,000	39,000		1.86
Louisville					
Jan.	6,000	7,000		1.15	
Feb.	4,100	4,200		1.02	
Mar.	8,200	7,400		.90	
Apr.	6,200	6,500		1.05	
May	5,900	5,900		1.00	
Jun.		3,800	4,100		1.08
Jul.		4,600	4,900		1.07
Avg				1.20	1.16*

* Abnormally high value of 4.31 omitted.

ency to follow the same variation pattern, approaching close agreement in most instances. This is in marked contrast to the low degree of reproducibility of MPN results, which tend to follow a course of wide variation in check samples. This contrast is exemplified by the fact that the normal variability of the MF count has been shown to be about 20 per cent, whereas the 95 per cent confidence limits for the 5-5-5 MPN count fall in the range of 30–300 per cent.

In the present study, a marked preponderance of tests showed higher MPN's than MF counts in the same samples, by a ratio of about two to one. But the tests during the 13-month period were made with one-step HD medium, which gave lower counts than other media used in later tests. If allowances were made for average performance by the two-step HD and one-step MF media, the counts recorded on the one-step HD medium would be increased by nearly 40 per cent (1.20×1.16). If, in addition, the MPN results recorded were adjusted to take into account the "bias" correction noted by Thomas and Woodward [1] to 81 per cent of observed values, the indicated preponderance of lower MF counts would undoubtedly be much less.

The remaining differences in the direction of higher MPN counts might, indeed, be attributable to the greater selectiveness of the MF count for *Esch. coli*, as Rawn and Bowerman [5] have suggested, or to the greater incidence of false positives in the MPN results, as suggested by another investigator. Or they may be overshadowed by the conclusion of Thomas and Woodward [1] that any observed deviations between MF and MPN results have little real significance in view of the low degree of reproducibility of MPN counts.

Past Investigations

A detailed review of the technique of MF examination of water, as first introduced into this country by Alexander Goetz in 1947, and subsequently modified by Kabler, Clark, and associates at the Taft Center, is contained in a 1953 task group report.[6] This report describes the apparatus required for the MF test, the media used for total bacteria counts, the modified EHC Endo * and a BGF media † used for coliform counts, and the detailed methods of handling the samples.

In a move to simplify the method of making the MF test, Hajna and Damon [2] proposed a single-step medium containing buffered sodium desoxycholate lactose broth as a substitute for such two-step procedures as in the use of EHC medium with a 2-hr enrichment process. This single-step medium, known as HD, is now available commercially, and has been used in the preliminary tests described in this article. Hajna and Damon gave the recommended formula of their proposed medium and full directions for its use.

Geldreich and associates [7] described a delayed incubation MF test for coliform bacteria in water. The water sample was filtered at the collection site, and the membrane was placed on a medium that preserves viability of the organisms and prevents multiplication during transit to the laboratory, where the viable bacteria can be cultivated and enumerated by routine procedures. To preserve the sample, sodium benzoate was added to EHC Endo medium. Detailed data were

* Proposed by USPHS Environmental Health Center.

† Containing ethyl alcohol (3 per cent) and brilliant green (0.00025 g/ml), in addition to the ingredients in the regular Endo nutrient.

presented showing the results of delayed MF counts compared to MPN's initially and on stored samples. Results showed that delayed MF incubation counts agreed well with MPN's on samples stored at 5°C, but tended to be less than initial MPN's. The delayed MF procedure is recommended for field use where shipment of refrigerated samples cannot be made, or during periods of emergency. Directions for the test were given in an appendix.

In 1956, Eye and associates [8] studied the effects of four different media and varying temperatures and humidities on results obtained in coliform enumeration by the MF procedure. At 32°C, consistently higher counts were produced on all four media than at 35°C. The highest average counts were given by the two-step Endo medium recommended in *Standard Methods* [3]; the next highest by the HD modified Endo medium. The best sheen was shown by the dehydrated scheduled nutrient (DSN) Endo, though the one- and two-step Endo produced a good sheen; the poorest was observed on the HD broth. The inhibitory effect of the DSN was excellent; the one-step Endo gave fair to good inhibition of noncoliform organisms. Results showed that coliform organisms may be grown on MF membranes at humidities greater than 35 per cent. The desired level of humidity may be obtained by subjecting a given surface area of water to evaporation with convection air circulation within a sealed incubator.

Clark and associates,[9] after having noted the acceptance of the MF method as part of the current federal drinking water standards, and after having described the provisions made in the standards for interpretation of the results, pointed out several common sources of error in technique: excessive number of colonies per membrane; elimination of enrichment period; low humidity during incubation; contamination of absorbent pads; flooding of membrane with culture medium; and air bubbles between membrane and absorbent pad. They stated that ideal membranes should have less than 60 sheen-producing colonies and not more than a 250–300 total colony count. But they added that good estimates can be made with 100 coliform colonies, when the noncoliform density is low. The authors then suggested ways to eliminate errors of technique. They noted that the presence of at least 1 ppm of zinc or copper, or both, in the water will inhibit the growth of coliform organisms on the membrane filter pads.

Several notable studies have been made on the advantages and limitations of the MF method of enumerating coliform organisms in water. The motive behind the development of the MF procedure has been the search for some process whereby the statistical difficulties and indirectness inherent in the standard dilution test could be eliminated, and a direct counting method involving no statistical complexities made available. This search began about 30 years ago, when direct plating methods on selective media were proposed. Such media as the Ayers-Rupp and, later, the Noble-Tonney were used. In one of a series of reports dealing with experimental studies of water treatment at the original Stream Investigation Station of USPHS in Cincinnati, the results of a comparative study of the Ayers-Rupp medium and the standard MPN method showed a high degree of agreement between the two. The main obstacle in establishing

direct counting methods for the coliform group has been in the definition of this group as lactose-fermenting, nonspore-forming, Gram-negative bacillus. Because "lactose-fermenting" was specified in this definition, the use of liquid media was virtually required, until Endo's solid medium became accepted as a confirmatory medium.

In a review prepared for the National Research Council and approved by the Division of Medical Sciences, Thomas and Woodward [1] made a detailed statistical analysis of the data from three studies. Defining the mean recovery factor as the ratio of mean MF to mean MPN, they showed that this factor for the overall data from the three studies was in the range 0.52–1.00, and nearly all the sampling stations had recovery factors in the range 0.3–3.00, which corresponds approximately to the theoretical range into which 95 per cent of replicate 5-5-5–tube MPN's may be expected to fall. Noting that the MPN count from the 5-5-5–tube test tends to be 23 per cent too high, they showed that if the MPN's observed in the tests were corrected for this "bias," a recovery factor of approximately 1.0 was obtained. In their summary, they stated that the disparity in coliform sensitivities of the MF and MPN in a wide variety of natural waters does not have much practical significance in view of the inherently low degree of reproducibility of the standard dilution test and the bias of MPN estimates. They further conclude that the precision attained with a single membrane filter was found to be two to five times greater than that of a 5-5-5–tube MPN. The reproducibility of the MF test was found to vary inversely with the square root of the number of colonies counted.

The same authors, with Kabler, discussed the use of membrane filters for water potability control.[11] After reviewing the points brought out in their previous article, they described a proposed coliform standard for drinking water based on the MF count, and summarized their findings by stating that the MF method has decisive advantages over the standard fermentation-tube method. They concluded that the adoption of the MF method as an alternative procedure for the measurement of coliform density would be a signal advance in sanitary engineering practice.

As a result of several years of experience with the membrane filter, Rawn and Bowerman [5] stated that as much as four times the hours spent for processing, and about three or four times the equipment, are required in the laboratory for the standard dilution test as are required for the same number of MF tests. Field inoculation of lactose tubes has long been practiced by these workers, but the membrane filter technique has proved much simpler and has involved practically no loss of samples.

Rawn and Bowerman also stated that the most controversial aspect of the membrane filter concerns the bacteriologic significance of the MF results as compared with the lactose fermentation test. Parallel testing, since early 1952, has revealed some numerical similarities, but also some perplexing anomalies. The question is posed as to whether the same organisms are counted in both tests. It is noted, however, that there is strong evidence that the membrane filter is selective for *Esch. coli* in mixtures of sewage and sea water. This is asserted to be a point in favor of the membrane filter. It is concluded that the simplicity, ease, convenience, precision, and speed with

which the MF test is completed, and the meaningful bacteriologic levels obtained, have been clearly demonstrated in the field and laboratory for about 4 years of routine testing by the Los Angeles County Sanitation Districts.

In a further appraisal of membrane filters, Eliassen [10] summed up the present status of this method of enumerating coliform organisms in water. He first noted that plate counts, decimal dilutions for the Phelps index, and, finally, the MPN technique "are in reality crude statistical approaches to the real number of organisms in a sample of water." Now, for the first time, he stated, sanitary science has a simple, accurate, and precise method of potable-water analysis in the MF technique. After analyzing the statistical shortcomings of the standard dilution test, Eliassen pointed out the advantages of the MF procedure, emphasizing that with this method, precision has been achieved far beyond that conceivable by any other practical method of coliform analysis.

Reported disadvantages of the MF method cited by Eliassen are: Turbidity, when present, may preclude the use of large samples; correlation with average MPN values may show lower numbers with MF counts; and toxic substances may influence the results. It is Eliassen's opinion that the advantages of the MF method far outweigh the disadvantages. He does not suggest dropping the fermentation-tube technique, as it will continue to be used by many analysts. But he does advocate adopting the MF method as another control method in *Standard Methods*.

Findings, Recommendations, and Action

The results of the comparative evaluation of the MF and MPN methods may be divided into the following:

Findings

1. The favorable agreement between MF and MPN determinations at most of the participating Orsanco laboratories has revealed the adaptability of the MF procedure for the bacteriologic examination of Ohio River water.

2. The precision of the MF count is greater than that obtained from the MPN procedure, and the MF count appears to offer a more accurate and sensitive means of enumerating the density of coliform organisms.

3. The MF technique provides results more quickly than the MPN procedure. Results are obtained in 18 hr with the MF technique, compared with 48 hr or more for the MPN method.

4. Overall operating costs, including labor and materials, for conducting the MF test range from about the same to one-third less than those for the MPN determination, and less incubator space is required.

5. The average agreement between the MF and MPN results at five plant laboratories of the six that completed the study ranged from 59 to 100 per cent during the second period of the tests, as measured by the individual results, and was 81 per cent for the six laboratories combined. Three of them reported an average agreement of 96–100 per cent for the same period.

6. Practice is needed to acquire the necessary skill for performing the MF test. For example, the average agreement between MF and MPN daily results for all the laboratories combined increased from 64 per cent for the first 6 months of comparative tests to 81 per cent during the next 7 months.

7. A troublesome source of error in the early MF tests was in the identifi-

cation of typical coliform colonies on the membrane. When the operator became more familiar with the identification of colonies and with the use of media permitting a sharper differentiation of typical colonies, the results greatly improved. Other factors that have not been fully evaluated in the present study, but which may lead to sources of error, are: effects of turbidity and algae, chemical characteristics of the water samples, and location of the sampling points with respect to sources of bacterial pollution.

8. From an operational viewpoint, the managers of the participating laboratories are unanimous in preferring the MF technique over the MPN test.

9. The most promising selective culture medium for coliform organisms, with respect to magnitude of recovery, appears to be the medium designated in this article as "one-step MF Endo."

Recommendations

The following recommendations were made:

1. The commission and the states signatory to the Ohio River Valley Water Sanitation Compact should approve the use of the MF procedure as an acceptable optional procedure to the standard MPN method for appraising the bacterial quality of Ohio River water.

2. Such approval should be conditioned on satisfactory comparative tests of the MF method in parallel with the standard MPN procedure, for a period of not less than 3 months. The method of measuring agreement shall be as followed in this article.

3. The approved method of conducting and interpreting the MF test should be as developed by the study group of the Orsanco Water Users Committee and as described in Appendixes A, B, and C of this report, with such modifications as may be adopted from time to time and duly approved by the signatory states.

Action

On Jan. 15, 1958, following the endorsement of the engineering committee, it was resolved that:

> The Commission adopt for its use, and recommend to the signatory states for their adoption, the membrane filter technique as an acceptable optional procedure to the most-probable-number (MPN) method for appraising bacterial quality of raw Ohio River water.

Summary and Conclusions

Under commission authorization and staff direction, members of the Orsanco Water Users Committee, during the period November 1956–November 1957, carried out a comparative study of the membrane filter and standard dilution methods of enumerating coliform organisms in raw waters taken from the Ohio River and two tributary rivers. Eight laboratories participated in the study, though the number was limited to six after the first 6 months. Before undertaking the study, the participants were given a brief indoctrination in MF technique at the Taft Center in Cincinnati.

Following completion of the 13-month period of comparative tests, the same group conducted a study of three different confirmatory MF media, in order to ascertain whether any improvement was obtainable over the one-step HD medium that had been used throughout the 13-month period of comparative tests. The first trial was the two-step HD medium against the one-step HD medium that had been previously used. This test continued for 5 months, from January through May 1958. An overall summary of the results revealed that the two-step HD

medium yielded an average of 20 per cent higher coliform counts than the one-step HD medium.

Meanwhile, interest had been aroused in a one-step medium proposed by Fifield and Schaufus.[4] Although only a limited time was available to test this medium, comparative tests against the two-step HD medium, during June and July 1958, showed an average of 16 per cent higher counts than the two-step medium—an encouraging result. It thus was indicated that substantially higher counts, amounting to nearly 40 per cent, were obtainable with the one-step MF medium as compared to the one-step HD medium.

The conclusions drawn from this study, made by six laboratories over a period of about 20 months, were that the use of the membrane filter is an acceptable alternative method of enumerating coliform organisms in water, as compared to the standard dilution method, and is preferable because of its precision, speed, economy of space, and its freedom from statistical complications.

It was indicated, however, that in order to obtain good results, the MF test must be made with careful attention to detail and with adequate training of the operating personnel. In this study, as long as 6 months was required before a general improvement in results was demonstrated. This period may be somewhat longer than that needed subsequently at other laboratories, as this was a pioneer effort by a group of laboratories, and time was required to adjust differences in procedure that affected the comparability of results. After these basic problems were worked out, there was much greater conformity. Nevertheless, practice periods as long as 3 months are needed to familiarize personnel with the procedures that will secure consistently good results with the MF method.

Finally, there are the questions of what section of the coliform group is measured by the MF test, and the extent to which the test is affected by excessive amounts of turbidity or algae. Regarding the first question, the consensus among qualified workers is that the MF test, when properly conducted, measures the section of the coliform group of greatest sanitary significance. Regarding the second question, which affects the mechanics of the test, only further experience under a wide variety of conditions can reveal the significance of the test under conditions found in the Ohio River valley. It appears, however, from experience gained during the 20-month period of the present study, that turbidity or algae should offer a source of difficulty for only limited and occasional periods of time. During periods of excessive turbidity or algae growth in the river, the dilution method can be used as an interim measure.

Acknowledgments

The investigation on which this article is based was carried out under the general direction of Edward J. Cleary, executive director and chief engineer of Orsanco. Authorization for the work was granted in a resolution of the commission dated Apr. 3, 1956. W. L. Klein, staff chemist-biologist, assisted in the field operations. Paul Kabler, chief of microbiology, and Harold Clark, of the microbiology section at the Taft Center, aided in training the participants and in providing counsel.

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APPENDIX A

Procedure for Comparative Study of MPN and MF Methods for Determination of Coliform Index

MPN Method *

A. Presumptive test with lactose broth or lauryl sulfate tryptose broth:

1. Sow five tubes on each of three dilutions.

2. Use sterile buffered dilution water for making dilutions.

3. Volumes of sample less than 1.0 ml should not be pipeted directly to the fermentation tube. Make a dilution such that 1.0 ml will contain the desired amount of original sample. For example:

Amount of Original Sample *ml*	Method of Sowing
0.1	1 ml from A (1:10) dilution
0.01	1 ml from A (1:100) dilution
0.001	1 ml from A (1:1,000) dilution
0.0001	1 ml from A (1:10,000) dilution

The following dilution guide accommodates the above range:

Dilution Desired	Original Sample	Sterile Buffered Dilution Water
1:10	1 ml	added to.... 9 ml
1:100	1 ml	added to....99 ml
1:1,000	1 ml of 1:100 dilution	added to.... 9 ml
1:10,000	1 ml of 1:100 dilution	added to....99 ml

B. Confirmed tests with 2 per cent brilliant green bile.

* Follow the procedure described in the tenth edition of *Standard Methods.*

C. Brilliant green bile tubes negative in 24 hr but positive in 48 hr to be confirmed by streaking EMB plates.

MF Method

A. Preparatory remarks:

1. The MF and MPN determinations should be made on the same sample or the same dilution. Do one and then the other.

2. Run at least two different volumes or dilutions on each sample. Volume selections should be such that not more than 60 coliform colonies are on the membrane. With an ideal volume selection, both plates can be counted, so that one is a check against the other. This is difficult to achieve practically when the coliform index fluctuates from day to day.

3. The recommended incubation period is 18 hr ±2 hr at 35°C. If operating conditions do *not* allow immediate counting at the end of the incubation period, place the plastic dishes in the refrigerator and count them within 4 hr. Do not attempt to compensate for the time factor by filtering and then allowing membranes to stand at room temperature before incubation.

4. Sterile dilution water for rinsing the funnel and sterile dilution blanks for use in making dilutions are prepared from the same formula. This is called "buffered dilution water" in the tenth edition of *Standard Methods*, and directions for preparing it are found on p. 367, Item 4.

5. Sterilize filter unit wrapped in special paper * at 121°C for 15 min in the autoclave.

6. Sterilize the packaged membranes and pads at 121°C for 10 min in the autoclave. Release the steam rapidly, and immediately remove the packaged units. Dry at room temperature.

7. The disposable plastic petri dishes are closed at high temperature in the process of manufacture. They may be assumed to be sterile as received for this study.

B. Equipment and materials. The following is based on the requirements for one sample in two different volumes:

1. One-liter filter flask connected to source of vacuum
2. Sterile filter funnel
3. Two sterile plastic dishes
4. Two sterile membranes
5. Two sterile absorbent pads
6. Forceps immersed in alcohol (95 per cent ethyl or pure methyl)
7. Sterile pipets (1-ml, 2-ml, and 10-ml) as needed, and sterile graduates when volume of single filtration exceeds 20 ml
8. Sterile dilution water for rinsing, and sterile dilution blanks (99-ml and 9-ml)

TABLE 4

Volume Recommendations

Size of Original Sample Desired *ml*	Add 1 ml Original Sample to 9 ml Dilution Blank	Add 1 ml Original Sample to 99 ml Dilution Blank	Diluted Sample *ml*	Condition
>10	no	no		Y
0.5–10	no	no		X
0.4	no	yes	40	Y
0.3	no	yes	30	Y
0.2	no	yes	20	Y
0.1	yes*	no	1	X
0.05	no	yes	5	X
0.01	no	yes	1	X

* This method of dilution is suggested in order that one dilution can be used for both MPN and MF methods.

* Kraft paper, a product of Albemarle Paper Mfg. Co., Richmond, Va.

9. M-HD medium
10. Plastic incubator container and cover, with moistened paper towel in bottom.

C. Procedure:

1. Flame forceps quickly for alcohol evaporation, not for sterilization.
2. With sterile forceps, place one sterile absorbent pad in each plastic dish. Replace forceps in alcohol.
3. Pipet enough M-HD medium to saturate each pad. Amount will vary 1.8–2.2 ml per pad. Tilt dish slightly and remove any excess with sterile pipet.
4. Flame forceps.
5. Transfer sterile membrane from package to filter receptacle (grid side up) with sterile forceps. Replace forceps in alcohol.
6. Measure water sample in funnel *with no vacuum* on filter flask.
7. Volume recommendation for good distribution (Table 4). Condition X: Pour about 1 in. of sterile dilution water into funnel before adding sample. Condition Y: Sample direct to funnel.
8. Turn on vacuum and filter sample.
9. With vacuum still on, rinse sides of funnel twice, using at least 20 ml of sterile dilution water each time.
10. Turn vacuum off. Remove funnel top and place on sterile paper (used in funnel sterilization) or on ring stand.
11. Flame forceps (as in No. 1), remove membrane from receptacle, and place membrane on M-HD impregnated absorbent pad. Avoid air bubble entrapment.
12. Place plastic dishes inverted on rack in plastic container. Secure cover and incubate at 35°C for 18 hr ±2 hr.
13. Count sheen colonies as coliform bacteria.

D. Preparation of M-HD Endo broth:

M-HD Endo broth is used in the membrane filter technique to culture coliform bacteria. This medium must be made up daily, because it cannot be made in quantities for storage. The number of grams of the dehydrated medium to be suspended in various quantities of cold distilled water is:

M-HD Endo Broth Dehydrated *g*	Cold Distilled Water *ml*
0.57	10
1.425	25
2.85	50

Weigh out the powdered medium and suspend in cold sterile distilled water contained in a sterilized 100-ml beaker. *Do not use buffered dilution water.* Mix with a sterile glass rod until the powdered material is thoroughly wet. Record the weight of media plus water. Heat to boiling with constant stirring. Do not boil for more than 5 sec. Allow to cool and make up loss in weight by adding more sterile distilled water. Do not autoclave. Cover with a sterile aluminum cap until ready for use.

APPENDIX B

Procedure for Interpreting MF Test Results

A review of the first sets of results received from the laboratories participating in the current comparative test of MPN versus MF for raw waters has raised some questions as to the methods followed in the MF tests and in the interpretation of the results. More specifically, these questions deal with the best way to plan the tests so as to give the optimum range of countable colonies on the filter pads, and with the methods whereby the proper counts should be selected for the final result from an individual test.

The optimum colony-counting range for the filter pads is about 20–80 colonies per pad. This range need not be too rigid—that is, a little variation is permissible when no count within the range is available. But the sample volumes filtered should be planned with a view to bringing the colony count as nearly as possible within this range.

Depending on the size of the counts from day to day, a sample volume may be selected to bring the colony count to about the middle of the range, about 50 colonies. Then, depending on the trend of the counts, a second volume is selected as about half or double the first volume selected. For example, if 1.0 ml is selected as the first volume, then 0.5 ml or 2.0 ml is the second volume, depending on whether the trend of the counts at the time was upward or downward. In some cases of doubt, both these volumes could be used as a second and third choice, though this should seldom be necessary.

With regard to the method of calculating the MF index for MF counts falling within the range of 20–80 colonies per plate, an expected variation of ±20 per cent can safely be assumed. This method, outlined below, should be followed in order that all results may be mutually comparable.

Rule 1

When conditions are ideal and the MF counts from two dilutions both fall within the range of 20–80 colonies, multiply the smaller result by the appropriate volume factor to have the results on an equivalent basis. Average the two results and convert the average figure to coliform index per 100 ml. For example:

Volume of Original Sample Used *ml*	MF Count
5	22
10	49

Then, 22×2 equals 44; the average of 44 and 49 is 47 (to the nearest whole number); and the coliform index per 100 ml is 47×10, or 470.

Rule 2

When the MF count from one dilution falls within the 20–80 range, and the MF count from a second dilution falls outside the 20–80 range, the former should carry the most weight. Multiply the MF count outside the 20–80 range by the appropriate volume factor to have the results on an equivalent basis. If the latter figure is within ±20 per cent of the MF count within the 20–80 range, average the

two figures, as in Rule 1. If the latter figure is not within ±20 per cent of the MF count within the 20–80 range, use only the MF count within the 20–80 range for calculation to coliform index per 100 ml.

Example A:

Volume of Original Sample Used *ml*	MF Count
1	12
4	75

Here, 75 is the key value; 20 per cent variation is 60–90 (75±15); 12×4 is 48. This last figure does not fall within the 20 per cent range above; discard result. Calculate the MF count of 75 to get the index: 75×25 becomes 1,875 per 100 ml. Express only *two* significant figures; 1,875 becomes 1,900.

Example B:

Volume of Original Sample Used *ml*	MF Count
1	15
4	75

Again, 75 is the key result; 20 per cent variation is 60–90 (75±15); 15×4 is 60. This last figure falls within the 20 per cent range above. Therefore, the average of 60 and 75 is 68 (nearest whole number). The coliform index per 100 ml is 68×25, or 1,700.

Example C:

Volume of Original Sample Used *ml*	MF Count
1	30
4	88

Here, 30 is the key result; 20 per cent variation is 24–36 (30±6); 88÷4 is 22. The last figure does not fall within the 20 per cent range above; discard result. Calculate the MF count of 30 to get the index: 30×100 is 3,000 per 100 ml.

Rule 3

When the MF counts from two dilutions both fall outside the range 20–80, the result nearest the 20–80 range should carry the most weight. Treat this figure as if it were in the 20–80 range, and apply Rule 2.

It will be seen from this method that the criterion for including or excluding a given count from an average is whether it falls within the optimum range (Rule 1) or within the ±20 per cent expected range of random variation of the "key result," which is within the optimum range of 20–80 colonies (Rule 2). In Rule 3, when both counts fall outside the optimum range, the one nearest this range would be selected as the final result. Thus, the three rules cover every instance that may arise.

APPENDIX C

Procedure for Comparative Study of MF Media

A. Two-step procedure using M-HD Endo medium with enrichment:

1. Lay out a series of three petri dishes.

2. Remove tops and place a sterile pad in each dish bottom.

3. Add 2 ml of M-enrichment broth to each pad.

4. Filter three different dilutions of the sample through respective membranes, and transfer the membranes, with sterile forceps, to the pads saturated with M-enrichment broth.

5. Invert the petri dishes and incubate them in the plastic container at 35°C for 2–3 hr.

6. Remove dishes from incubator. Remove tops and place in an inverted position. Using sterile forceps, lift each membrane and its absorbent pad to the inverted tops of the petri dishes.

7. Pour any excess M-enrichment broth from the bottom half of the dishes.

8. Place a new sterile absorbent pad in the bottom of each dish.

9. Saturate the pads with 2 ml of M-HD Endo broth.

10. Using sterile forceps, transfer the membranes to the new absorbent pads containing M-HD Endo broth. Discard the absorbent pads containing M-enrichment broth.

11. Incubate for 16–18 hr at 35°C in the plastic container.

B. One-step procedure using M-HD or MF Endo media:

1. Sow the same three dilutions for each of the above media as was used for the enrichment method.

2. Proceed as outlined in Appendix A.*

C. Preparation of media:

1. M-HD Endo broth. Prepare as outlined in Appendix A.

2. M-enrichment broth.

a. Dissolve 5.4 g of the dehydrated medium in 100 ml of distilled water.

b. Sterilize in the autoclave for 15 min at 15 psi.

c. This medium may be kept for relatively long periods, provided excessive evaporation does not take place.

3. MF Endo broth.

a. Dissolve 4.8 g of the dehydrated medium in 100 ml of distilled water containing 1.0 ml of ethanol.

b. Heat to boiling.

c. Do not prolong boiling or heating in autoclave.

* It is suggested that the membranes for the comparative methods be prepared at the same time, in order that the same dilutions may be used for each method. Enrichment will create a lag period for the two-step procedure. Therefore, the final incubation period for this procedure will be completed 2–3 hr after the end of the incubation period for the one-step procedure.

Continuous-Flow Water Pasteurizer for Small Supplies

Melvin Goldstein, L. J. McCabe Jr., and Richard L. Woodward

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IN general, uncontaminated ground water from properly constructed wells is the most economical source of safe drinking water for the individual rural home. In many areas, however, ground water of suitable quality and quantity is not obtainable, and cisterns or farm ponds are used as supply sources. There is a need for inexpensive treatment units requiring little operation and maintenance for these water users.

Disadvantages of Chlorination

Disinfection is necessary where surface water is used. Ground water of dubious bacterial quality, such as is often found in cavernous limestone wells, also needs disinfection. Although chlorination is the most common water disinfection process, it presents the problem of feeding a measured dosage. The need for continual servicing and inspection and the cost of good feeding devices are the principal disadvantages of the process. Because high chlorine residuals make the water unpalatable, there may be temptation to maintain the residual at a low level with only a small safety margin, thus taking a risk when the contact time following dosage is short or when chlorine demand varies. Experience has shown that only a small percentage of such chlorinators have been operated continuously and with sufficiently frequent measurement of chlorine residuals to assure adequate disinfection.

Most householders lack the time, ability, or inclination to service a mechanism of any degree of complexity. A simpler disinfection method, which can be easily checked or controlled, is needed.

Pasteurization

The success in killing pathogens in milk by pasteurization suggested the use of heat as a means of eliminating the problems of chemical feeding in a small water treatment device. The application of heat is the oldest and best understood of disinfection processes. Variations in chemical impurities in water do not interfere with the disinfection efficiency of heat as they do with chlorine and many other chemical disinfectants. Equipment for automatic control of heat processes is available, relatively inexpensive, and reliable. In addition to bacterial pathogens, heat readily destroys such other organisms as amebic cysts, worms, and viruses, which are more resistant to chemical disinfectants than vegetative bacteria.

The current temperature and time recommendations for milk pasteurization are 161°F for 15 sec or 145°F for 30 min. The high-temperature, short-time process was chosen for study to eliminate the problem of storage.

For pasteurization to be economically feasible, it is necessary to use the heat of the treated water to warm the water to be treated. It is also necessary to operate the unit at a nearly steady flow rate to reduce heat losses due to frequent starting and stopping of flow. Thus a storage reservoir from which treated water is pumped or flows upon demand is required.

A study was made of the total cost of pasteurization for various electric-power rates, daily water usages, and heat exchangers, based on list price of equipment. Because assembly is a comparatively simple job, the cost was not included. Results of the study were promising and a unit was designed for treating enough water to meet the daily needs of a family of five based on a usage of 50 gpcd (250 gal per 12 hr). A larger unit (1,000 gal per 12 hr) was also designed.

Pilot Plant Investigations

A 250-gal per 12-hr unit was built and operated to test the performance of the various components and the overall efficiency and economy of the process.

Figure 1 is a schematic diagram of the water pasteurizer. Untreated water enters the heat exchanger, where its temperature is raised to about 151°F by exchange of heat from the treated water flowing counter-current to the incoming water. The bypass pump pumps the water from the heat exchanger through an electric heater, which raises the water temperature to the pasteurization temperature of 161°F. A 30-ft, ¼-in. copper retention tube assures at least a 15-sec holding time at that temperature. An air relief valve permits escape of gases liberated at the higher temperature.

A solenoid valve and thermostatic switch are so arranged that if the water temperature falls below 161°F after the

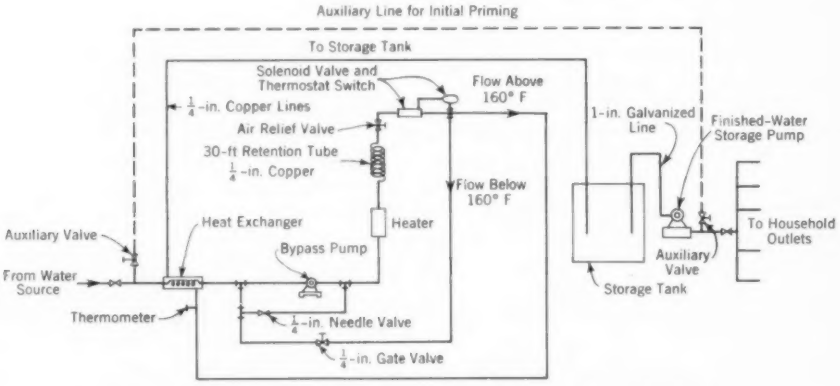

Fig. 1. Flow Diagram of Continuous-Flow Water Pasteurizer

Approximate prices of the major components of the unit are given in Table 1.

water passes through the retention tube, the water is returned to the suction side of the bypass pump and recycled through the heater. When the water at the thermostatic switch is 161°F or higher, the water flows to the heat exchanger, where it gives up its heat to the incoming water and is cooled to within about 10°F of the temperature of the incoming water before it enters the storage tank. A pressure pump is also used to provide water to the household plumbing system unless the storage tank is at an elevation sufficient to supply water by gravity flow.

The auxiliary line shown in Fig. 1 supplies water for priming of the system. The line can be a hose or other temporary connection and should not be connected permanently to the system.

The heat exchanger was the most difficult component of the unit to procure commercially at a reasonable cost. Commercial liquid-liquid exchangers are made for higher flow rates and for higher pressures and temperatures than would be encountered in the proposed service and are, therefore, more costly than necessary for this unit. Refrigerator condensers are better suited to the flow and pressure requirements of the proposed system, but they are designed for gas-liquid exchange, have unbalanced areas, and are not usable with a unit using the same liquid on both sides of the exchanger.

Several economical designs for heat exchangers were conceived and tested. The design finally selected has performed satisfactorily and is relatively inexpensive and easy to construct. The exchanger was made by the insertion of a ¼-in. Type K soft-temper copper tube inside a ½-in. Type K soft-temper copper tube (Fig. 2). Preliminary tests indicated a heat transfer coefficient of 199 Btu/hr/sq ft per degree Fahrenheit, and a heat exchanger 107 ft long was constructed. Figure 2 is a photograph of the heat exchanger. List prices of the materials totaled $55. A similar 100-ft exchanger was made by a local contractor for $85. To reduce heat losses, a box was constructed for the exchanger and both the exchanger and the retention tube were covered with granular insulation.

TABLE 1

Major Components of Pasteurizer Unit and Approximate Prices *

Item	Price $
Heat exchanger	95.00
Bypass pump	69.60
Heater	54.50
Air relief valve	6.50
Thermostatic switch	15.00
Solenoid valve	14.50
Thermometer	12.50
Finished-water storage tank	95.00
Finished-water pump	83.50
Float switches	15.00
Retention tube	5.00
Well for thermostat	5.08
Valves	10.90
Fittings	10.50
Pump suction line	1.80
Wiring	21.98
Total	516.36

* Early 1958; a detailed list of components and their manufacturers may be obtained from: Director Robert A. Taft San. Eng. Center, 4676 Columbia Parkway, Cincinnati 26, Ohio.

The pump used serves both as a bypass and main pumps (Fig. 3). Most pumps available for the small design flow of 0.35 gpm are not built for continuous service, and some difficulty was experienced in obtaining a suitable pump.

As electric power is available in most rural areas, an electric heater—the smallest convenient one that would fulfill design requirements and reduce

heat losses to a minimum—was selected. A commercial unit consisting of a 1,500-w heater inside a 2-in. insulated pipe was chosen. The first unit was constructed and put into operation in early February 1958, in the experimental wing of the Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio. A second all-electric unit was installed at Point Pleasant, Ohio, in August 1958, with a farm pond as the source of supply. In January 1959, a third unit using a liquid petroleum gas for heat was constructed at the Taft Center.

The unit at Point Pleasant, Ohio, is constructed on the same principles as the all-electric unit at the Taft Center, but different equipment was selected to perform equivalent functions. Because results on the latter two units have not been fully evaluated, the tables included in this report cover only the unit constructed in February 1958. Preliminary data indicate that the gas unit is only half as efficient as the electric units in energy utilization.

The first unit, diagramed in Fig. 1, has been found satisfactory to date.

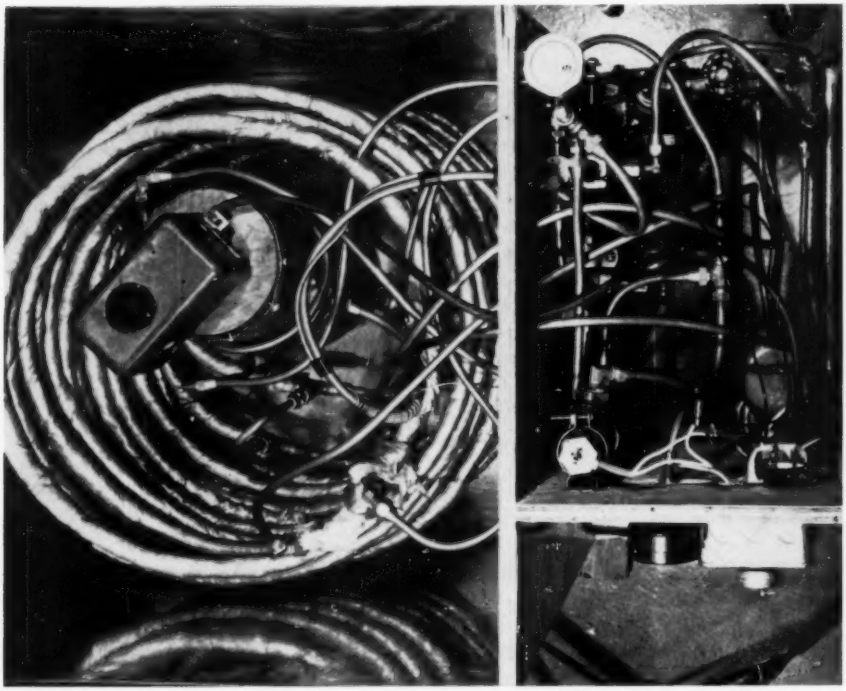

Fig. 2. Top View of Pasteurizer Unit

The large insulated coils on the left are the heat exchanger tubes. The heater and coiled copper retention tube are visible inside them. The thermostatic switch, pump motor, air relief valve, and major valves are in the right-hand compartment (Fig. 3).

The major components and their list prices (early 1958) are given in Table 1. The assembled unit is shown in Fig. 4.

Objectives of Pilot Studies

For proper evaluation of performance data, the following objectives of the pilot plant studies should be defined:

1. To confirm the effectiveness of the unit in killing bacteria
2. To measure the power consumption of the unit

TABLE 2

Removal of Coliform Organisms by Slow Sand Filtration and Pasteurization, 1958*

Date of Samples	Max. Treated-Water Temperature °F	Retention Time at Max. Temperature sec	Density in Raw Water MPN/100 ml	Density in Clear Well MPN/100 ml
Jan. 22	166	2	1,600	540
27	164	2	110,000	1,400
29	166	2	14,000	2,300
Feb. 3	165	2	7,900	110
19	164–174	2	4,900	130
24	164–176	2	1,700	33
26	164–176	2	7,900	490
Mar. 10	166–176	2	4,900	3,300
12	164–176	2	79,000	3,300
17	164–176	2	460,000	700
19	164–178	2	23,000	330
24	162–170	2	1,400	130
Apr. 7	164–179	15	13,000	79
9	168–178	15	1,700	13
14	161–170	15	11,000	13
16	165–175	15	2,200	220
21	161 170	15	490	6.8
23	161–170	15	33,000	79
May 5	163–170	15	460	49
12	164–177	15	22,000	100
21	164–171	15	11,000	4.5
26	168–180	15	1,100	33

* No coliform organisms were found in any of five 10-ml portions per sample examined after pasteurization.

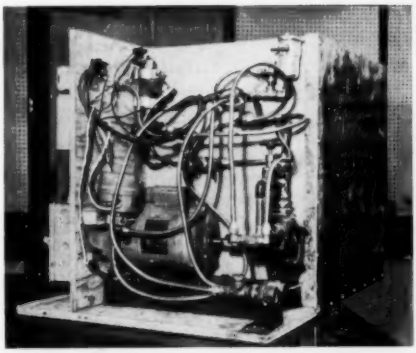

Fig. 3. Side View of Pasteurizer Unit

The valves, pump, and pressure switch are housed in this compartment.

3. To determine maintenance requirements and reliability of the equipment
4. To check the significance of scale or corrosion in reducing the operating efficiency or life of the equipment.

Destruction of Coliform Organisms

Removal and destruction of coliform organisms by filtration and heat disinfection are shown in Table 2. A 2-sec retention time was used before the 30-ft holding tube was inserted in the system. At no time during normal operation were coliform organisms isolated from the finished water. On one occasion the unit was challenged by adding coliform organisms to the clear well to produce a concentration of 950,000 per 100 ml. Portions of 50 and 100 ml of the treated water were tested by the membrane filter technique. One coliform organism was isolated from the 50-ml portion, for an average count of 0.75 per 100 ml. Thus, even with a very high bacterial concentration in the unpasteurized water, the final product was satisfactory.

An electric-power meter installed on the line to the unit measured the total power required for pumping and heating. In early February 1958, pasteurization was started at an average production rate of 250–270 gpd. The unit was in almost continuous operation, except for weekends, for 360 days. During this time, 53,400 gal was treated. Data on power consumption (Table 3) are shown for the first 3 months and for the final month of operation. The average power consumption per 1,000 gal was 37.7 kw-hr. At a cost of 1 cent per kilowatt-hr, the cost per 1,000 gal would be 37.7 cents. This cost is approximately 50 per cent higher than initially predicted. One factor contributing to the difference is the lower power efficiency of small pumps, giving an actual power cost about ten times the theoretical cost. Another factor is the heat loss during starting. The total cost of water based on a 10-year amortization of equipment and a yearly consumption of 90,000 gal, is 97.7 cents per 1,000 gal. This cost does not include any maintenance or servicing charge.

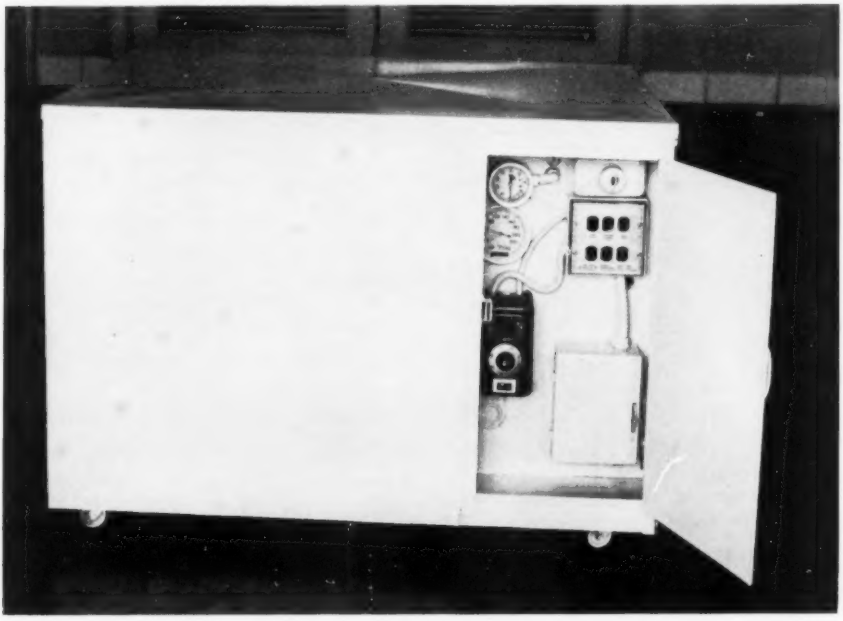

Fig. 4. Front View of Assembled Pasteurizer Unit

The instruments to the right include temperature and pressure gages, the main switch box, the pilot light, manual switches, and the thermostat control setting.

Equipment Performance

The only piece of equipment to give performance trouble was the pump on the first unit. Four pumps tested have

failed to give satisfactory performance. On one, the carbon brushes on the motor and the carbon vanes on the pump impeller wore quickly. On others, leaks developed which required frequent service.

The turbine vane pump now in use has given approximately 18 months' maintenance-free service to date and the equipment promises to be suitable for continuous use.

Corrosion and Scale

Little Miami River water, which is pumped into the experimental wing of the Taft Center, was treated by the

TABLE 3

*Operating Costs of Water Pasteurization Unit, 1958–59**

Month and Day	Water Used *gal*	Electrical Power Used *kw-hr*	Cost of Electrical Power† ¢	Cost of Treated Water‡ *¢/1,000 gal*	Month and Day	Water Used *gal*	Electrical Power Used *kw-hr*	Cost of Electrical Power† ¢	Cost of Treated Water‡ *¢/1,000 gal*
Feb. 17	270	10.6	10.6	39.1	Apr. 16	250	10.0	10.0	40.0
18	250	9.9	9.0	39.5	17	250	8.9	8.9	35.6
19	250	9.2	9.2	36.6	18	280	10.8	10.8	38.5
20	250	9.6	9.6	38.3	21	250	9.5	9.5	38.0
21	260	9.7	9.7	37.3	22	310	11.1	11.1	38.5
24	250	8.6	8.6	34.4	23	290	10.7	10.7	36.8
25	250	8.6	8.6	34.4	24	290	10.7	10.7	36.8
26	250	9.3	9.3	37.1	25	260	9.1	9.1	35.0
Mar. 5	250	8.6	8.6	34.4	May 1	250	9.3	9.3	37.2
9	250	10.3	10.3	41.0	2	250	9.8	9.8	39.2
10	250	9.5	9.5	38.0	Dec. 29	290	9.6	9.6	33.0
11	250	9.5	9.5	38.0	30	250	9.1	9.1	36.4
12	260	9.3	9.5	38.0	31	210	7.7	7.7	36.6
13	250	9.5	9.5	38.0	Jan. 2	160	6.0	6.0	37.5
14	250	10.6	10.6	42.4	5	270	10.5	10.5	38.5
17	250	7.9	7.9	31.5	6	280	11.1	11.1	39.5
18	250	9.5	9.5	38.0	7	240	9.4	9.4	39.0
21	270	10.0	10.0	37.2	8	300	11.7	11.7	39.1
23	270	9.9	9.9	36.5	9	250	10.1	10.1	40.4
Apr. 1	250	10.3	10.3	41.2	12	250	9.4	9.4	37.6
2	250	9.4	9.4	37.6	13	260	10.0	10.0	38.5
3	250	9.0	9.0	36.0	14	270	10.2	10.2	37.8
4	290	10.7	10.7	36.9	15	280	10.5	10.5	37.5
7	250	9.0	9.0	36.0	16	270	11.5	11.5	42.5
8	250	9.5	9.5	38.0	21	250	9.6	9.6	38.5
9	250	9.3	9.3	37.2	26	250	11.2	11.2	44.8
10	250	9.1	9.1	36.4	27	240	9.2	9.2	38.3
11	250	9.0	9.0	36.0	28	280	11.0	11.0	39.3
14	280	10.7	10.7	38.2	29	290	11.5	11.5	39.7
15	280	10.8	10.8	38.2	30	290	10.6	10.6	36.5

* Rate of flow was 0.35 gpm; inlet water temperature was 47°–61°F; pasteurization temperature was 162°–179° F.

† Based upon an assumed rate of 1 cent per kilowatt-hour.

‡ Average cost was 37.7 cents per 1,000 gal.

TABLE 4

Sample Chemical Analyses of Water Before and After Pasteurization

Item	Concentration—*ppm*	
	In Influent†	In Effluent‡
Calcium (Ca)	74	74
Magnesium (Mg)	23	23
Chloride (Cl)	14.3	14.3
Sulfate (SO_4)	56.3	55.8
Alkalinity (as $CaCO_3$)	205	207
Hardness (as $CaCO_3$)	268	268
Dissolved oxygen	5.63	3.34
Carbon dioxide*	21	19
Copper (Cu)	0.03	1.04

* By titration.
† pH 7.72.
‡ pH 7.76.

pasteurizer during the first 6 months of its operation.

Table 4 shows typical chemical analyses of the influent and effluent water. The only significant change in chemical quality during passage through the pasteurizer was a slight increase in its copper content, a common occurrence in new copper plumbing. Copper dissolution decreased as the unit continued to operate. Inspection of test sections has shown no evidence of significant corrosion or scaling.

Conclusions and Recommendations

The pasteurization unit described in this report has performed satisfactorily in destroying coliform organisms. Because there is no residual disinfecting action following heat treatment, it is essential to avoid recontamination of the water in storage or in the plumbing system.

The total cost of water pasteurization on a household scale is estimated to be approximately $1 per 1,000 gal. This cost is high compared with that of chemical disinfection. The principal advantages of pasteurization are its reliability and simplicity. The major mechanical difficulty encountered to date has been in finding a suitable pump. The pump presently in use may solve this problem. Scaling and corrosion problems have not been important.

Additional field testing of the units is desirable. Drawings of the units and instructions for constructing the heat exchanger are available; requests should be addressed to: Director, Robert A. Taft Sanitary Engineering Center, 4676 Columbia Parkway, Cincinnati 26, Ohio.

Acknowledgment

The pasteurization unit was conceived by Samuel R. Weibel and Kenneth H. McCorkle, Jr. The ingenuity of Arthur H. Turner, mechanic, contributed immensely to the development of the unit.

Suggested Method for Nitrate Determination With 2,6-Xylenol Reagent

Arnold M. Hartley and Richard I. Asai

A contribution to the Journal by Arnold M. Hartley, Asst. Prof., and Richard I. Asai, Research Asst., both of the Dept. of Chemistry & Chem. Eng., Univ. of Illinois, Urbana, Ill. *The investigation was supported, in part, by Grant S-75 from the National Institutes of Health, USPHS.*

IN *Standard Methods*,[1] two methods are given for the determination of nitrate ions in water: the phenoldisulfonic acid method and the reduction method. In recent years, the use of brucine as a colorimetric reagent for nitrate has been reinvestigated and proposed for the determination of nitrate ions in water.[2, 3] The brucine method has the advantage over the standard methods in speed and simplicity. The brucine color system does not obey Beer's law, and, owing to variability of color development from one occasion to another, one must run a set of standards along with the sample.

A new spectrophotometric method, employing 2,6-xylenol as a reagent, is suggested for the determination of nitrate ions. Absorption measurements are made at 322–324 mμ. Beer's law is obeyed in the concentration range of 0–125 ppm nitrate, and precision, expressed as the coefficient of variation, averages about 1 per cent in this range. Direct determination of samples is possible and requires less than 10 min.

1. General Discussion

1.1. *Principle.* 2,6-Xylenol is known to react with excess nitric acid to form 4-nitro-2,6-xylenol,[4] the only mononitro derivative reported in the literature. The conditions under which nitrate ion and excess 2,6-xylenol reagent react quantitatively to form the 4-nitro derivative have been investigated. For aqueous samples of nitrate, two acid mixtures were found to be satisfactory as nitrating media; these are 6:3:1 sulfuric acid–water–acetic acid and 4:4:1:1 sulfuric acid–phosphoric acid–water–acetic acid mixtures by volume. The 10 per cent acetic acid is present as the solvent for the reagent. In these acid systems the reaction between nitrate and 2,6-xylenol is complete within 5 min. The procedure calls for the addition of 1.00 ml of an aqueous nitrate sample and 1.00 ml of 2,6-xylenol reagent to 8.0 ml of an acid mixture. After a reaction time of approximately 5 min, the absorbance of the reaction mixture is measured at 322 or 324 mμ, depending upon the acid system employed.* For a single spectrophotometer, cell, stock solution, and type of sample, the concentration-absorbance line is linear and constant. Therefore,

*For reasons beyond the scope of this article, these wavelengths do not correspond to the maximum absorbance of the known 4-nitro-2,6-xylenol.

unknown nitrate concentrations may be determined from the experimental molar absorptivity. Spectral curves of $1.001 \times 10^{-3}M$ 2,6-xylenol, $1.00 \times 10^{-4}M$ 4-nitro-2,6-xylenol, and a reaction mixture containing 14 μg nitrate nitrogen and $10^{-3}M$ 2,6-xylenol reagent are shown in Fig. 1.

1.2. *Interference.* Nitrite and chloride were found to interfere with the method. Both ions cause large recoveries of nitrate but are easily elimi-

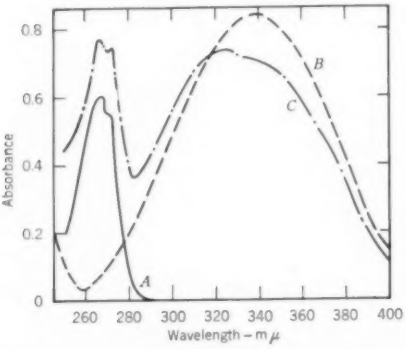

Fig. 1. Spectral Curves of Reagents
Curve A *represents* 1.001×10^{-3}M *2,6-xylenol;* B, 1.00×10^{-4}M *4-nitro-2,6-xylenol; and* C, *nitration mixture (14.0 μg nitrate nitrogen) in 6:3:1 sulfuric acid–water–acetic acid.*

nated. The addition of solid sulfamic acid to reduce nitrite to nitrogen, satisfactorily eliminates the nitrite interference. Chloride interference is eliminated by adding solid silver sulfate (low in nitrates) and removing the precipitated silver chloride by filtration.*

2. Apparatus

2.1. *Spectrophotometer,* suitable for use at 322–324 mμ.

2.2. *Quartz absorption cells,* 1-cm.

2.3. *Buret,* 50-ml, equipped with special plastic † stopcock ‡ to dispense highly acid mixtures.

3. Reagents

3.1. *2,6-Xylenol* (0.01*M*). Dissolve 1.2216 g of 2,6-xylenol,§ and dilute to 1 liter with glacial acetic acid.

3.2. *Stock nitrate solution* (100 ppm nitrate nitrogen). Dissolve 0.7221 g of analytical reagent grade potassium nitrate, and make up to mark of 1-liter volumetric flask with distilled water.

3.3. *Acid mixtures.* Mix equal volumes of concentrated sulfuric acid and 85 per cent phosphoric acid (both reagent grade), or three volumes of concentrated sulfuric acid and one volume of distilled water. Cool to room temperature.

4. Procedure

4.1. *Construction of standard curve.* Prepare nitrate standards from stock nitrate solution, by proper dilution, to be 2.0, 4.0, 8.0, 12.0, and 16.0 ppm in nitrate nitrogen. From a buret equipped with a special plastic stopcock, add 8.0 ml of either acid mixture to a 50-ml erlenmeyer flask. Pipet 1.00 ml of the aqueous nitrate standard and 1.00 ml of stock 2,6-xylenol reagent into the flask. Gently swirl the contents of the flask and, after allowing a 4–5-min reaction time, measure the absorbance of the solution at 322 mμ if the H_2SO_4-H_2O acid system is employed, or at 324 mμ if the H_2SO_4-H_3PO_4 acid mixture is em-

* Further details and data can be found in the Ph.D. thesis of R. I. Asai, University of Illinois (1959).

† Teflon, a product of E. I. du Pont de Nemours & Co., Wilmington, Del.

‡ Manufactured by Fischer & Porter Co., Hatboro, Pa.

§ No. 1772, from Eastman Organic Chemicals Department, Rochester, N.Y.

TABLE 1

Nitrate Determination in Known Samples With the 2,6-Xylenol Method

Nitrate Nitrogen Added μg	Absorbance* at 324 mμ	Nitrate Nitrogen Found μg	Coefficient of Variation
2.87	0.162	2.88	2.17
5.73	0.321	5.71	1.25
11.5	0.646	11.3	0.96
17.2	0.963	17.1	0.57
22.9	1.288	22.9	0.10
28.7	1.616	28.8	0.51

* Average of five determinations.

ployed. Correct for reagent blank and construct standard curve. For a single spectrophotometer, cell, stock solution, and type of sample, the concentration-absorbance line is linear and constant. Therefore, unknown nitrate concentrations may be determined from the experimental molar absorptivity.

4.2. *Determination of nitrate in aqueous sample.* Take an aqueous sample containing 0–16 ppm nitrate nitrogen (dilute if necessary). If more than 10 ppm chloride is present, treat sample with solid reagent grade silver sulfate (low in nitrates), and remove precipitated silver chloride and excess silver sulfate by filtration. A large excess of silver sulfate is tolerable. If nitrite is also present, treat filtrate with solid sulfamic acid.* Proceed as in 4.1. From absorbance reading at 322 or 324 mμ, either determine nitrate concentration from standard curve or calculate from experimental molar absorptivity.

5. Interpretation of Results

5.1. The method was applied to samples of known nitrate concentrations. To 8.0 ml of 1:1 sulfuric acid–phosphoric acid mixture were added 1.00 ml of nitrate standard and 1.00 ml of stock 2,6-xylenol reagent. The absorbance measurements at 324 mμ, measured against nitrate concentration, were linear over the range of 0–125 ppm nitrate ion. The precision of the method is excellent, as shown in Table 1.

5.2. The nitrate concentration in several ground and river water samples was then determined (Table 2). The samples contained nitrate in the concentration range of 0–10 ppm, which required the introduction of a concentration step. After treating the samples with solid silver sulfate and removing the precipitated silver chloride by filtration, 10.0 ml of the filtrate was evaporated to dryness in a 50-ml erlenmeyer flask by the use of an infrared lamp. It was found that the pH of the aqueous sample, prior to evaporation, was very critical. At a pH greater than 7, the residue was slightly charred, and subsequent absorption measurements were slightly erratic. At a pH less than 6, loss of nitrate occurred. After evaporation of the samples to dryness, 8.0 ml of the acid

TABLE 2

Nitrate Determination in Ground and River Water Samples

Sample No.	No. of Determinations	Nitrate ppm	Average Deviation ppm
1	4	2.69	0.02
2	4	2.85	0.02
3	4	3.15	0.06
4	4	7.46	0.11
5	4	1.69	0.06
6	3	3.43	0.09
7	4	3.78	0.13
8	4	3.42	0.11

* No. 4659, from Eastman Organic Chemicals Department, Rochester, N.Y.

mixture, 1.00 ml of water, and 1.00 ml of 2,6-xylenol reagent were added to the flasks. The contents were gently swirled, and after a 5-min reaction time the nitrate concentration was determined from absorbance measurements at 324 mμ.

From preliminary experiments, there seems to be a good indication that 3.00-ml aqueous samples can be employed for a direct determination, as outlined in the procedure section. Extreme caution must be taken, as considerable heat is evolved by the addition of 6.0 ml of concentrated sulfuric acid to 3.00 ml of water. This would allow one to determine lower levels of nitrate without evaporating the sample.

References

1. *Standard Methods for the Examination of Water, Sewage, and Industrial Wastes.* APHA, AWWA & FSIWA, New York (10th ed., 1955). p. 149.
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Determination of Carbonate Alkalinity in Natural Waters

J. F. J. Thomas and J. J. Lynch

A contribution to the Journal by J. F. J. Thomas, Head, and J. J. Lynch, Scientific Officer, both of the Industrial Waters Section, Mineral Processing Div., Mines Branch, Dept. of Mines and Technical Surveys, Ottawa, Ont.

WATER supply chemists and engineers, who are primarily concerned with natural or unpolluted waters for domestic and industrial use, usually define alkalinity as that property of an aqueous solution which enables it to neutralize acidity to the equivalence point, thereby increasing the hydrogen-ion concentration and decreasing the pH of the aqueous solution. They usually consider only "carbonate" alkalinity—that is, dissolved bicarbonates, carbonates, and hydroxides—and report it in equivalents, or parts per million, of calcium carbonate. In polluted waters, industrial brines, and industrial waste waters, alkalinity, or basicity, may be due not only to carbonates but to weak organic acids and other anions, such as silicates, borates, and phosphates. Analysts and others concerned with such waters may neutralize to other than the equivalence point. It is assumed, for the purpose of this discussion, that if these other basic anions are present, in the water under study, they are so only in insignificant amounts in comparison with the carbonate present.

In natural waters fundamental equilibria exist between free carbon dioxide, bicarbonate, carbonate, hydroxide, pH, and total alkalinity. These relationships have been studied by many workers and applied to various aspects of water analysis and treatment.[1-6] From these fundamental relationships the ionic concentrations of bicarbonate, carbonate, and hydroxide can be calculated if the pH and total alkalinity are known. The accuracy of the calculations depends, therefore, largely upon the accuracy with which the pH and total alkalinity are determined.

Carbonate Alkalinity Determination

Standard, or referee, methods for the determination of alkalinity are well documented.[7, 8] The water sample is titrated with a standard solution of a strong acid (hydrochloric or sulfuric acid) to the equivalence points of the first and second dissociations of the carbonate ion.

The methods differ principally in the means employed to determine the endpoints, the accuracy of the method depending upon the reliability of the latter determination. In standard, or referee, procedures the equivalence points or endpoints are determined by electrometric titration, and in routine plant methods they are determined by the use of organic indicators. Various indicators or mixtures of indicators

have been recommended and used by analysts to increase the accuracy of the visual determination of endpoints and to give closer agreement with the true equivalence points.[8-11]

The most accurate procedure, and one that is also suitable when other than carbonate alkalinity is present, is to determine the total carbon dioxide content of the water.[5] From this value and the pH of the water, the relative amounts of each ion contributing to carbonate alkalinity can be calculated from theoretical equations. The method is, however, time consuming, and it demands such careful technique that it is not generally suitable for routine use.

Larson and Henley developed a method for determining low alkalinities in rain water which permitted not only greater accuracy in determining the true equivalence point but decreased the error due to the sliding endpoint common in weakly alkaline waters.[12] They found that for the low alkalinities present in rain water (0–7 ppm as $CaCO_3$) a plot of hydrogen-ion concentration (or activity) versus volume of standard acid became a straight line after the true equivalence point was reached. Extrapolation of this straight line to a hydrogen-ion concentration of zero moles per liter $[(H^+) \times 10^{-7} = 0]$ determined the amount of acid required at the equivalence point (endpoint) and thereby the total alkalinity of the water. They assumed that extrapolation should be made to $(H^+) \times 10^{-7} = 1$, or pH 7, but showed that extrapolation to $(H^+) \times 10^{-7} = 0$ introduced negligible error. They recommend the method only for waters having low alkalinity

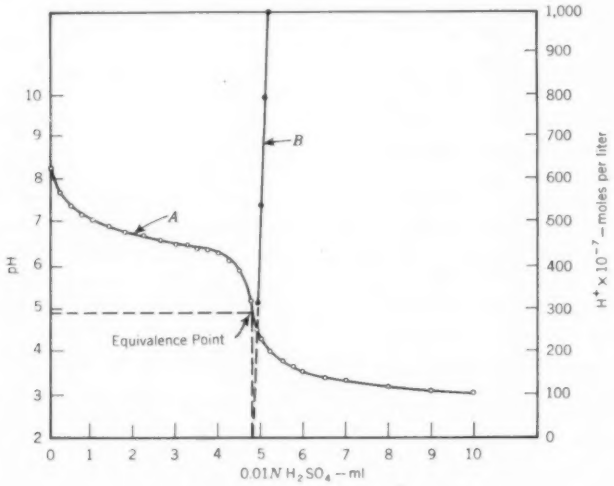

Fig. 1. Determination of Equivalence Point

Data are for sample No. 9686. Curve A *has the pH scale as its ordinate; the ordinate for Curve* B *is the molar concentration (right-hand scale).*

or acidity. Their procedure, although accurate for such waters, is quite time consuming.

Equivalence Point

It has been recognized for some time that the true equivalence point pH for dissociation of the bicarbonate ion to carbonic acid varies with the total alkalinity.[5, 10, 13]

Calculation from fundamental equations shows that the inflection point, or equivalence point pH, for the first-dissociation (carbonate to bicarbonate) is relatively constant at pH 8.35–8.36, so that titration to pH 8.3, as is normally required in standard methods, is sufficiently accurate.

Similar calculations and the technical literature, however, show that the equivalence point pH for the second-dissociation (bicarbonate to carbonic acid), whereby the total alkalinity is determined, varies with the amount of alkalinity present, the variation being particularly significant at low alkalinities. Recently developed referee methods for the determination of alkalinity have recognized this by recommending titration to varying pH endpoints, such as 5.1, 5.0, 4.8, 4.5, 3.5, depending upon the total alkalinity. These methods, although they decrease the error, do not eliminate it and cannot be applied without some prior knowledge of the total alkalinity.

It is also important to realize that both equivalence points vary with temperature and that the error may be significant unless temperature is controlled within ± 5°C during alkalinity determinations.

In the course of other studies, it was found necessary to determine accurately the alkalinity of a large number of waters in which the alkalinity was due to carbonates and bicarbonates. The procedure developed by Larson and Henley was studied because it was claimed that it eliminated the error due to varying equivalence points. The procedure outlined below and adapted from that method was found to be accurate and sufficiently rapid to be used routinely in the laboratory for natural waters. Studies are continuing on the application of the procedure to other waters and aqueous solutions.

TABLE 1

Comparison of Methods for the Determination of Total Alkalinity

Water Sample or Sodium Bicarbonate Solution	Alkalinity (as $CaCO_3$)—*ppm*	
	Larson and Henley Procedure	Calculated From Total Carbon Dioxide
$NaHCO_3$ solutions		
1	28.0	27.5
2	83.3	82.5
3	95.3	96.5
4	127.4	126.9
5	141.2	141.9
Water samples		
M 548	71.0	71.2
M 550	89.8	89.7
M 551	90.1	89.8
M 560	15.6	15.7

Procedure

The equivalence point pH was determined for 65 natural waters, sodium carbonate solutions, and sodium bicarbonate solutions having total alkalinities ranging from 0.5 to 400 ppm (as $CaCO_3$).

Titration curves similar to Curve *A*, Fig. 1 were determined for each water or aqueous solution using a pH meter.*

* Model N; manufactured by Beckman Instruments, Fullerton, Calif.

The procedure of Larson and Henley was then applied to the data (see Curve *B*, Fig. 1 and 3) and the straight-line portion of these plots extrapolated to the abscissa [$(H^+) \times 10^{-7} = 0$]. Perpendiculars drawn from these intercepts with the abscissa were considered to intersect Curve *A* of each titration curve at the equivalence point pH, or endpoint, of the titration.

Equivalence point pH values were then plotted against total alkalinities of the waters calculated from the volume of standard acid used at the intersection to produce the curve shown in Fig. 2. The dotted line at pH 3.9 (Fig. 2) marks the methyl orange endpoint used by Schroeder.[5] *Standard Methods,*[8] considered the methyl orange endpoint to be at pH 4.3. Both values are significantly lower than the true equivalence point pH values.

The validity of the application of the Larson and Henley method over this wide range of alkalinities was tested as follows:

1. Total alkalinities, determined for a number of the waters, in the manner described above, were compared with alkalinities calculated from determinations of total carbon dioxide content of the same waters.

2. Alkalinities, determined by the Larson and Henley method, for aqueous solutions of cp grade sodium bicarbonate were checked by determining the equivalents of sodium ion in the same solutions.

3. The equivalence points obtained by the Larson and Henley procedure were compared with those calculated from theoretical equations and with those obtained from a graphical determination of the inflection point of each electrometric titration.

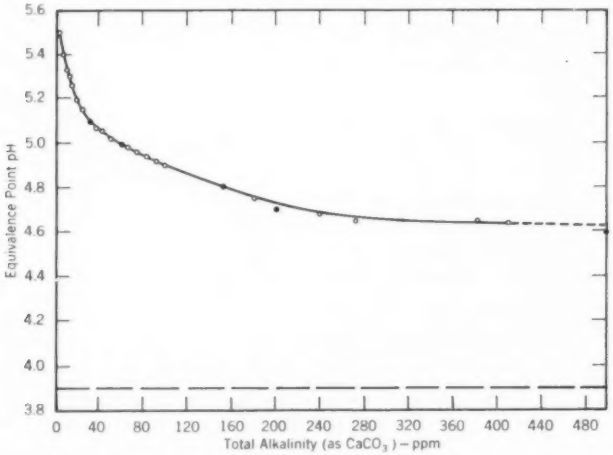

Fig. 2. Variation of Equivalence Point pH With Total Alkalinity

Solid circles represent values calculated by Dong, Guisti, and Greenberg[10]*; open circles are those of the authors.*

As the above studies were all carried out at room temperature and at normal atmospheric pressure, whereas Larson and Henley titrated alkalinities while bubbling a carbon dioxide–free gas through the water, tests were also carried out under the latter conditions.

Results

In Table 1 alkalinities determined by the Larson and Henley method and by the total carbon dioxide evolution method are compared. In view of the errors in determining the line slope graphically instead of mathematically and those inherent in the carbon dioxidle evolution method, the two procedures show satisfactory agreement.

Table 2 confirms the validity of the application of Larson and Henley's procedure to highly alkaline water. In this table the equivalents per million (epm) of bicarbonate ion as determined by this procedure agree well with the equivalents per million of sodium when the method is applied to known aqueous solutions of chemically pure sodium bicarbonate.

Figure 4, a plot after the Larson and Henley procedure for natural waters over a range of alkalinities, indicates some variation in the slope and length of the straight-line portions (Curve *B*, Fig. 1 and 3). Such variations are mainly due to the volume of water sample titrated and the amount of total alkalinity.

Figure 5 is a plot of the rate of change of pH with tiny increases in acid. Such a plot graphically determines the inflection point, or equivalence point, of the titration. In view of the error inherent in this method, because special care was not taken to maintain the acid increments very small, agreement between it and the Larson and Henley method is considered very close (Table 3). It will be noted that very small changes in acid volume (1 drop) are magnified when converted to equivalence point pH or alkalinity, especially in the steepest portion of Curve *A* (Fig. 1).

Routine Use of Method

The method described above is very time-consuming, as it necessitates a plot for each water and graphical determination of the intercept.

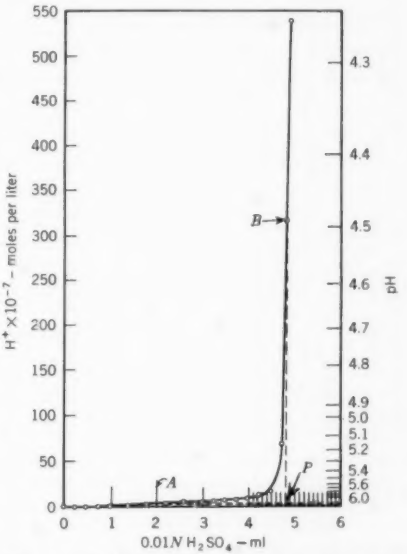

Fig. 3. Total Alkalinity Determined by Larson and Henley Procedure

Data are for sample No. 9686.

Figure 6 is a replot of a section of the graph of Fig. 3. If Points *A* and *D* are taken on the straight-line portion of the Larson and Henley plot, then, from Fig. 6,

$$X_0 = X_1 - (X_2 - X_1) \times \left(\frac{Y_1}{Y_2 - Y_1}\right) \quad . . (1)$$

in which X_0 is the intercept with the X-axis, or the volume of acid required to neutralize to the equivalence point.

If $A(X_1, Y_1)$ and $D(X_2, Y_2)$ are chosen so that $Y_1 = 316 \times 10^{-7}(H^+)$, or pH 4.5, and $Y_2 = 631 \times 10^{-7}(H^+)$, or pH 4.2, then Eq. (1) becomes

$$X_0 = X_1 - (X_2 - X_1) \times \left(\frac{316 \times 10^{-7}(H^+)}{[631 \times 10^{-7}(H^+)] - [316 \times 10^{-7}(H^+)]} \right) \quad (2)$$

which reduces to $X_0 = X_1 - (X_2 - X_1)$, in which X_1 and X_2 are the volumes of acid required to titrate to pH 4.5 and pH 4.2 respectively and $(X_2 - X_1)$ is the increment of acid required to decrease the pH from 4.5 to 4.2.

Other pH values may be used and other equations developed, but the length of the straight-line portion of the plot varies with total alkalinity (Fig. 4), and, as it is necessary that both pH or H^+ values lie on that straight line, the choice of pH endpoints is limited.

As the points X_1 and X_2 are taken 0.3 pH units apart, choice of the points at pH 4.6 and 4.3 or 4.4 and 4.1 will result in the same value for X_0 and, therefore, alkalinity. Thus, precise standardization of the pH meter is not required as in titration to a particular pH endpoint.

For many waters the straight-line portion of the plot (Curve *B* in Fig. 1 and 3) is sufficiently extended to permit further titration to pH 4.0. The increment of acid required to titrate from pH 4.2 to 4.0 should be about 1.17 times that required from pH 4.5 to pH 4.2. This additional titration serves as a check on the previous titration and the acid increment $(X_2 - X_1)$.

TABLE 2

Validity of the Larson and Henley Method by Determination of the Ionic Balance

Sodium Bicarbonate Solution No.	Larson and Henley Procedure		Flame Photometer Method		Error *per cent*
	Alkalinity as $CaCO_3$ *ppm*	Alkalinity as HCO_3 *epm*	Sodium *ppm*	Sodium as HCO_3 *epm*	
6	4.2	0.0836	1.9	0.0826	−0.6
7	8.9	0.1773	4.1	0.1783	+0.3
8	18.2	0.3640	8.5	0.3696	+0.8
9	28.3	0.5655	13.0	0.5652	−0.1
10	69.6	1.1407	26.3	1.1435	+0.1
11	85.6	1.7095	39.7	1.6957	−0.4
12	114.4	2.2864	52.6	2.2871	0.0
13	142.7	2.8519	65.6	2.8523	0.0
14	170.1	3.3993	78.3	3.4045	+0.1
15	199.4	3.9844	91.6	3.9828	0.0

Recommended Procedure

For natural waters containing mainly carbonate alkalinity the usual referee electrometric-titration method for alkalinity may be carried out with a relatively accurate pH meter and volumetric buret.

The sample of water (25 to 100 ml) is titrated at room temperature (20°–30°C) to pH 4.5 and the required volume (X_1) of standard acid recorded.

The titration is carefully continued to pH 4.2 and the required volume of acid (X_2) is again recorded. The increment of acid from pH 4.5 to 4.2 ($X_2 - X_1$) is noted and subtracted directly from the volume used at pH 4.5 (X_1) to give the amount of acid required to neutralize the alkalinity to the true equivalence point of the bicarbonate–carbonic acid dissociation. The total alkalinity is then calculated in the usual manner—that is,

$$\text{alkalinity} = \frac{1{,}000f[X_1\text{ ml} - (X_2\text{ ml} - X_1\text{ ml})]}{\text{ml sample}}$$

or

$$\frac{1{,}000f}{\text{ml sample}}(2X_1\text{ ml} - X_2\text{ ml})$$

This procedure was applied to aqueous solutions of anhydrous sodium carbonate with alkalinities of 25, 50, 100, 200, and 500 ppm (as $CaCO_3$), the titration being carried out four times on each solution without agitation with carbon dioxide–free gas.

The reproducibility of the method under routine laboratory conditions was found to be very satisfactory. This method is considered to be more accurate than present referee procedure, as it eliminates both the error due to variation of the equivalence point with total alkalinity and the need for precise pH meter standardization. It is still sufficiently rapid for routine use.

Discussion

Tables 1–3 and Fig. 1–3 confirm the validity of extending the Larson and Henley procedure to a wide range of alkalinities. Also, the equivalence point pH determined by the Larson and Henley procedure over a wide range of alkalinities produced a curve (Fig. 2) which corresponds with the curve developed from theoretical equations by Dong, Guisti, and Greenberg.[10]

Figure 3 is a complete plot following the Larson and Henley procedure of the titration curve of one water sample (No. 9686). It is seen that a portion of the graph below pH 6.5 [$(H^+) = 3 \times 10^{-7}$] is also a straight line (A), which, upon extrapolation, intersects the extrapolated Line B at Point P.

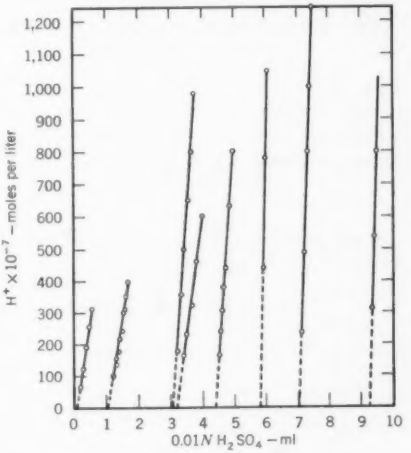

Fig. 4. Extrapolation Curves for Natural Waters of Varying Alkalinities

The extrapolation curves, plotted from results of the Larson and Henley procedure, vary somewhat in slope and length; this is mainly due to differences in the volume of sample titrated and the total alkalinity.

This point is considered to be the true intercept from which the perpendicular to the titration curve should be drawn in order to determine the equivalence point pH.

Intersection of these straight lines (P) is at $X = \frac{b_1 - b_2}{m_2 - m_1}$ where the respective equations for the straight lines, A and B, are $y = m_1x + b_1$ and

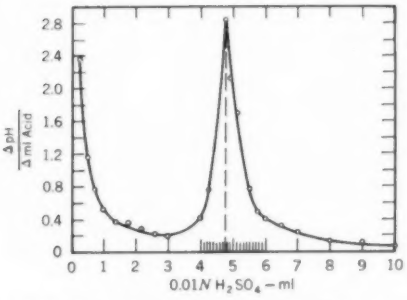

Fig. 5. Graphical Determination of the Inflection Point

Data are for sample No. 9686. In view of the error involved in this method, agreement between the graph and the results of the Larson and Henley procedure is quite clear.

$y = m_2x + b_2$. The intersection of Line B with the X-axis ($y = 0$) is $x = \frac{-b_2}{m_2}$ so that the error in extrapolating Line B to the abscissa is

$$\frac{b_1 - b_2}{m_2 - m_1} - \frac{-b_2}{m_2} \quad \text{or} \quad \frac{m_2b_1 - m_1b_2}{m_2^2 - m_1m_2}$$

This error is small, as b_1 and m_1 are very small in comparison to b_2 and m_2; for example, for sample No. 9686,

$$\frac{b_1}{b_2} = 4.4 \times 10^{-5} \text{ and } \frac{m_1}{m_2} = 4.9 \times 10^{-4}.$$

It is evident that Larson and Henley were applying these principles and were not titrating, as they assumed, to an equivalence point of pH 7 for all waters.

A comparison was made of the results obtained when applying the Larson and Henley technique of bubbling the water with carbon dioxide–free gas with the procedure employed in this study (electrometric titration under atmospheric conditions). Differences between the two techniques were very small over a range of natural alkalinities 16–250 ppm as $CaCO_3$; there was, however, some indication that agitation by carbon dioxide–free gas increases the alkalinity slightly in the lower range and decreases it in the higher ranges. With titration to pH 4.5 the presence of 20 ppm carbon dioxide would cause an increase in pH of only 0.005, which is well within the limit of error of the pH determination. It is recognized that greater accuracy should result if uniform conditions of agitation with carbon dioxide–free air are employed, as any small variation in equivalence endpoint due to the carbon dioxide remaining at the end of the titration is eliminated.

As the Larson and Henley procedure is dependent on pH, as a measure of the hydrogen-ion activity rather than concentration, a series of tests were made on waters having abnormally high salt contents. By this

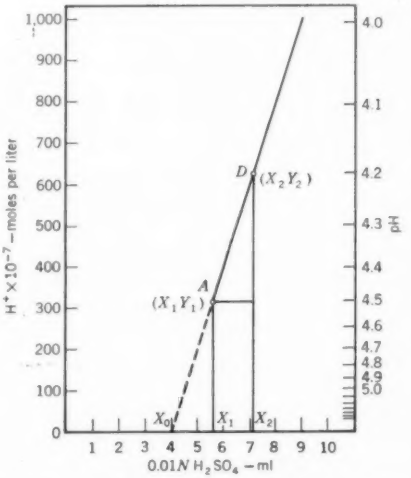

Fig. 6. Derivation of the Modified Larson and Henley Procedure

This is a replot of a portion of the graph in Fig. 3, $X_2 - X_1$ is the volume of acid required to reduce the pH from 4.5 to 4.2.

procedure alkalinities of sodium carbonate solutions of 20, 100 and 400 ppm alkalinity to which had been added 1,000 ppm sodium chloride were determined. No significant effect was noted when the procedure was applied to these highly saline solutions.

It is therefore concluded that the Larson and Henley procedure determines very closely the true equivalence point of the titration and that it can be applied satisfactorily to natural waters having a wide range of alkalinities.

Adaptations of this technique to determinations of acidity, and to visual determinations of alkalinity when suitable indicators are available, are under study.

Conclusion

The method of Larson and Henley has been shown to be applicable for waters having a wide range of alkalinities. The method increases the accuracy of the determination of carbonate alkalinity as it eliminates the error due to variation of true equivalence point with total alkalinity. The method determines the actual inflection point or equivalence point pH of the titration, and it has been adapted to routine use with no significant decrease in accuracy. The variation of equivalence point of the titration of bicarbonate ion to carbonic acid is graphically shown over a wide range of alkalinities.

TABLE 3

Equivalence Points Determined by the Larson and Henley and Graphical Methods

Sample No.	Larson and Henley Method			From a Graphical Determination of the Inflection Point		
	Acid at Equivalence Point *ml*	Equivalence Point pH	Total Alkalinity (as $CaCO_3$) *ppm*	Acid at Equivalence Point *ml*	Equivalence Point pH	Total Alkalinity (as $CaCO_3$) *ppm*
9704	0.10	5.42	0.5	0.20	5.18	1.0
9660	1.05	5.40	5.4	1.10	5.25	5.7
9737	3.20	5.16	16.5	3.25	5.05	16.7
9697	3.04	5.12	31.3	3.10	4.90	31.9
9678	4.38	5.10	45.3	4.45	4.93	45.8
9689	3.57	5.00	73.5	3.50	5.15	72.1
9686	4.80	4.90	98.9	4.75	5.18	97.9
9699	5.75	4.75	148.1	5.70	5.09	146.8
9709	5.79	4.80	149.1	5.80	4.78	149.4
9702	7.01	4.75	180.5	7.10	4.62	182.8
9701	9.21	4.68	237.2	9.20	4.75	236.9
9747	11.33	4.66	291.7	11.30	4.75	291.0
9748	12.21	4.62	314.4	12.20	4.65	314.2
9723	13.25	4.67	341.2	13.15	4.90	338.6
9711	14.83	4.65	381.9	14.75	4.75	379.8

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Installation and Maintenance of Gate Valves in St. Paul

Roy W. Holzer

A paper presented on Sep. 17, 1959, at the North Central Section Meeting, Minneapolis, Minn., by Roy W. Holzer, Supt. of Distribution, Water Dept., St. Paul, Minn.

AMONG the principal reasons that gate valves are needed in a distribution system, probably the most important is to control, with ease and speed, the flow of damaging water when a break occurs in the mains. Another important use of valves is to close off a portion of the system when new cut-ins are being installed. Furthermore, the dividing gate is needed to segregate different pressure areas and still make it simple to reunite the systems when difficulties arise. The many other uses for valves—controlling air vents, blowoffs, hydrants, and service connections—will not be discussed.

Valve Spacing

With regard to a main break in the distribution system, the question of how many valves would normally have to be closed brings to mind the important problem of valve spacing. Although a designer can put on paper an ideal system layout, it is doubtful that many ideal arrangements would be discovered in actual use. The problems of arriving at a nearly perfect valve system are many. The cost, the tying into an old existing setup, or, perhaps, a change in the entire system has to be considered. It seems, therefore, that emphasis should be given to a valve installation scheme that would be both economical and workable. Such a scheme necessitates many considerations. It depends, for example, on whether the pertinent area is in the business district, the industrial section, or the residential part of the city. Figure 1 shows part of the distribution system in a typical residential area in St. Paul, Minn. This portion of the system is 0.5 mi on each side, which is also the average grid spacing for the city. The grid mains across the top and bottom of the figure are 12 in. in diameter, the one at the right is 16 in., and the main at the left is 12 in. Also supplying this area are seven horizontal 6-in. service mains and one 8-in. secondary grid main, running vertically in the figure. The gate valve spacing in this typical area, although not perfect, is economical and very workable. The left and right portions of the figure show that every horizontal main has gate valves on each side of the vertical grid main. This, too, is practically the setup around the secondary 8-in. grid main. According to the figure, some gates have not been installed; of course, these would be added when the system is improved.

The grid mains are gated at least every half mile, and then on both sides of the intersecting grid. In some instances, the grid mains are gated approximately every 1,300 ft. This is often desirable when secondary grid

lines intersect. With this arrangement of valves, no matter where a service main break occurs, only two blocks, or approximately 1,300 ft of main, would be affected. In most instances, the closing of two valves would control the break. If trouble developed along the vertical grid mains, many valves would have to be closed, but this difficulty cannot be easily avoided. It would be well to mention that larger mains, which are used on the grids, usually split or crack, thus requiring more time for repairs. As more than 90 per cent of the consumers are served from the other mains, no serious problem develops. The area shown in the figure is common in most cities, but new, modern plats have resulted in many problems related to the distribution system and the spacing of valves. Each problem requires special study.

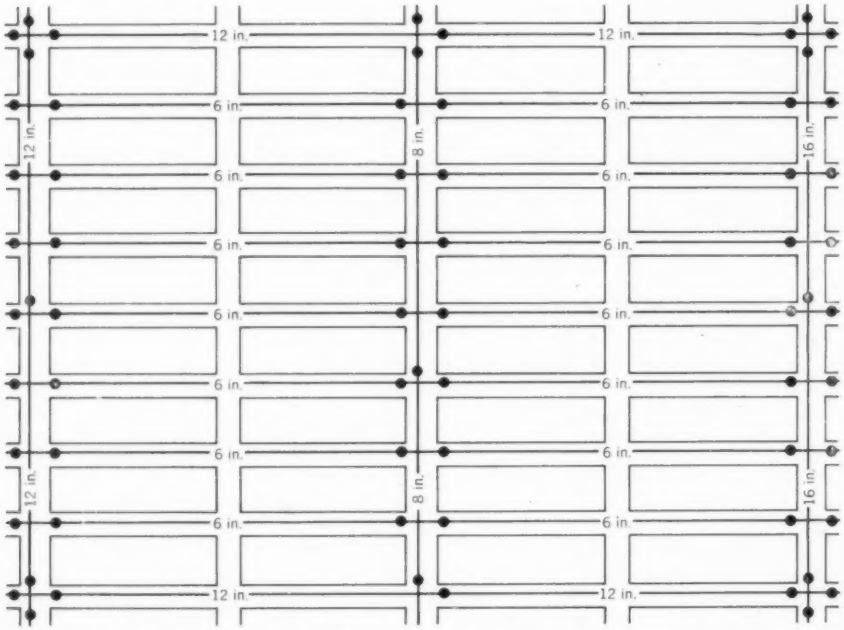


Fig. 1. Portion of Distribution System in a Typical Residential Area in St. Paul

The solid circles shown on the mains represent the gate valves. With this valve spacing, regardless of where a break in the main occurs, only two blocks (about 1,300 ft of main) are affected. The illustration is not drawn to scale.

Types of Valves

Each city or community has a standard policy on the type of gate valves and control appurtenances used in the distribution system. The system that is used in St. Paul is an economical one and is also quite satisfactory from a maintenance viewpoint. For several years, St. Paul has used the AWWA type of valve with the nonrising stem. The gates have iron bodies with double-

disk, bronze, mounted parallel seats, and open to the right. The only exception to the AWWA standard is the greater tensile strength (80,000 psi) required in the valve stem. In other words, St. Paul requires the same stem strength for valves less than 30 in. as AWWA requires for larger gates. The city's present policy is to use the vertical type of gate valve with a cast-iron gate box for 16-in. and smaller size valves; for gates 20 in. and larger, the horizontal type of valve with a bypass is used. For this type of installation, a precast concrete manhole is set around the gate mechanism, and a cast-iron gate box is used for the bypass valve.

Tapping valves, for a number of reasons, are being used a great deal more at present. In the business and industrial areas of St. Paul's distribution system, it is almost impossible to shut down for new construction during the ordinary working week. With the use of tapping valves and sleeves, however, construction can proceed without inconveniencing consumers or decreasing fire protection in the area. The policy of using existing crosses and tees that were previously installed in the system is followed wherever feasible. The tapping valves are installed frequently when provisions have been omitted for the usual fitting type of cut-in. This situation is occurring more often, because new plats of streets frequently result in mains connected to existing lines at unsatisfactory locations. Another factor responsible for the increased use of the tapping valve is the excellent type of air-driven tapping machines now available. These have decreased installation costs immensely. At present, St. Paul taps into 4–42-in. mains. The tapping sizes of the branch lines, however, range from 3 in. to and including 12 in. Standard gate boxes are used with all sizes of tapping valves.

The three-piece gate box set over vertical valves is a very serviceable item if installed correctly. The box consists of a separate base or pot designed to fit over the bonnet and operating nut of the vertical gate. Bases of various size are made to fit valves of corresponding size. The middle piece is a standard section that fits into all bases and has a threaded portion near the upper end. The top section of the box is also threaded to fit over the middle piece and to allow for grade adjustment. A suitable cover fits flush into the top section and, when properly set, forms a flat surface along the street roadway. During installation of a gate box, a few simple precautions must be taken. First, the soil must be firmly compacted around the bonnet of the valve to prevent uneven settlement. In questionable soil, brick or creosoted timber may be needed under the base. The box should be well centered over the operating nut and must be set vertically with a carpenter's level. If these simple tasks are accomplished during installation, no operating difficulties should be experienced.

Manholes for Valves

The manholes that are constructed around the mechanism of the horizontal gate valves are approximately 4.5 ft in diameter and set on a concrete footing. The bottom of the manhole consists of the natural soil to allow for drainage. No attempt is made to house the entire bonnet of the gate valve, as experience has shown that its removal is not required often enough to justify the installation of a larger manhole or vault. The main purpose of the manhole is to allow inspection, lubrication,

and access for packing the valve, as well as to provide an excellent means for the operation of the gate. At present, the precast concrete sections with precast cones on top are the most satisfactory from a standpoint of economy and ease of construction. The sections may be purchased in various lengths, so that the depth of the gate valve does not present a problem for the manhole builder. A standard metal frame and cover is used to top off the manhole. Sewer brick or curved concrete blocks also make excellent manholes for gate valves, but the labor involved makes them expensive.

Maintenance

St. Paul is divided into four approximately equal districts, with a water service foreman in charge of each. It is his responsibility to check every gate valve in his district as often as time permits. It takes about 2 years to inspect each area. This inspection consists of locating and checking each gate box for correct distances to property lines, for proper grade, and, if the box is clear, for operation. Service crews are not authorized to operate the gates. Deficiencies, like packing leaks, are repaired by authorized crews. In addition, a great amount of maintenance is accomplished when valves are operated during main breaks and new construction cut-ins. Of course, this is precisely when perfect operation of valves is desired, but the discovery of defects at these times has the effect of emphasizing to foremen and crews the importance of valve maintenance every hour of every working day.

Gate valve maintenance is a complex problem. Attempts are being made to divide maintenance into various categories, with the more important valves getting the most attention. The gates on the larger feeder mains, and on those smaller ones that must be operated in conjunction with them, are checked at least once each year. This inspection consists of locating and measuring the distance from each manhole or gate box to the property line, making those adjustments necessary because of possible grade changes in the street, or adding extra asphalt material. Nothing can be so time consuming, when frost is in the ground, as the task of removing a few inches of frozen asphalt from the top of a manhole or box. Next on the checklist is to make certain that the gate box or manhole is not filled with earth or stone, which often prevents operation of the valves. Finally, the gate is actually operated, during which process most of the valves are completely closed. Where large valves cannot be shut without endangering the system, they are closed only to the halfway point. A followup crew immediately packs and makes other minor but necessary repairs. This inspection and maintenance procedure covers about 20 per cent of the city's 10,000 gate valves. The other valves are not forgotten, but less time is devoted to their maintenance.

Jacking of Long Pipe Section Beneath a Highway

Henry Goldfinger

A contribution to the Journal by Henry Goldfinger, Project Mgr., Spencer, White & Prentis, New York, N.Y.

JACKING a large-diameter pipe through a fill usually presents no special problems when the length of the pipe is less than 60–70 ft. With longer pipe, unforeseen field problems often arise, which may necessitate the completion of a job by more expensive methods. This happened recently when one contractor had to abandon jacking because of excessive buildup of pressures and had to complete his job with a liner plate tunnel.

It becomes more difficult to hold a large-diameter pipe to an accurate line and grade as its length increases. Buckling of the pipe is another problem as the total jacking force rises. Time of shutdown between jacking shifts must also be considered to avoid unnecessary increase of jacking pressures.

Installation of Steel Shell

An awareness of the critical factors and proper design aided a contracting company to pass a ½-in. thick, 66-in. diameter steel shell beneath a six-lane highway (Fig. 1) in Queens, N.Y. The job was done without mishap or settlement of the road above. The 102-ft pipe was held to a 1-in. line and grade tolerance without difficulty, and cleared a 48-in. overhead sewer by 8 in. at the receiving end.

Although the contractors were prepared to sink a shaft at midpoint and jack from both ends, this was not necessary. Sufficient jacking force was made available to push the pipe through without excessive deviation; the pipe was made heavy enough to withstand buckling. The steel pipe acted as a cylinder through which a 48-in. concrete water line passed. The space between pipes was later grouted.

Experience was the principal factor in estimating the maximum jacking pressures in advance. The soil under the highway, previously deposited by a hydraulic fill, is coarse sand underlaid with fine sand at the jacking level. A resistance of 250–350 psf to overcome the friction through this material was anticipated. Except for the first 25 ft, these values checked out very closely in the field, varying between 250 and 500 psf. Jacking pressures and unit frictional resistance of the soil were high at the outset, the unit frictional resistance decreasing as work progressed.

A 27×10-ft pit was dug on the north side of the road and sheeted with 3-in. boards for the its entire 17-ft depth. One tier of 12-in. bearing-pile (BP) 53-lb steel wales were placed all around, 7 ft, 6 in. below the surface. Water was encountered 9 ft down,

and wellpoints were installed by the general contractor to keep the pit dry. Pumps were placed in an adjoining pit.

The principal tools used in the jacking operation consisted of a pair of hydraulic jacks * with 44-in. strokes and a cradle made from a wide-flange (WF) steel beam. Each of the jacks had a 200-ton capacity, a total of 400 tons—about twice the pressure finally needed. The cradle was made from a 13-ft length of 36-in. WF 230-lb beam, placed with its flanges vertical. To facilitate sliding the steel pipe, lengths of slit pipe, 2.5 in. in diameter, were placed on the two upper flanges. The beam was secured on 12×12-in. timbers.

Jacking of the pipe and hand mucking of the soil were carried out simultaneously. The jacking force was applied along the horizontal center line of the pipe—12 ft, 6 in. below the surface—to two 15×60-in. steel plates, 1 in. thick, stiffened by 12-in. I beams that distributed the force around most of the pipe's periphery. When the jacks completed a stroke, they were retracted, and short lengths of I beams were placed between them and the pipe to fill the space of the advance. The jacks were extended again. Five strokes pushed one 16-ft length of pipe into the ground. One foot of pipe re-

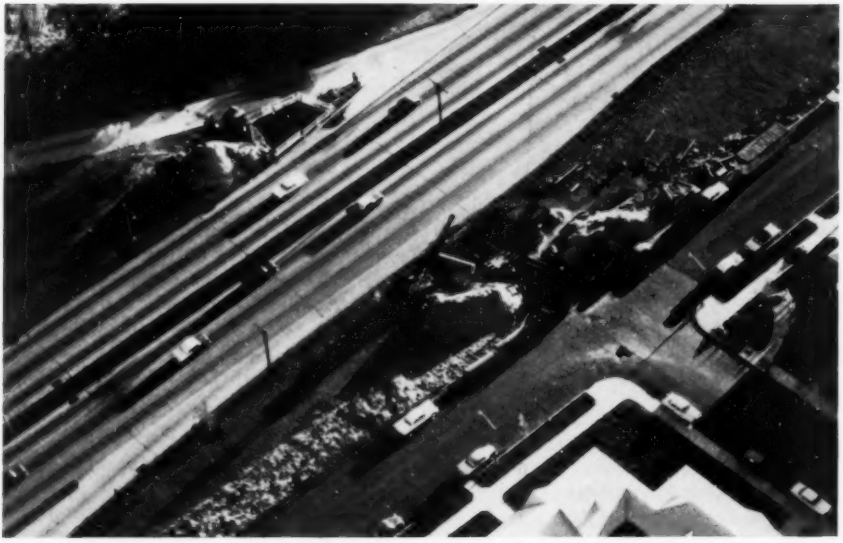


Fig. 1. Aerial View of Site of Jacking Operations

Shown in the photograph are the two pits dug opposite one another, on each side of the highway, one pit near a feeder lane. The 102-ft section of pipe was passed under the highway without disrupting traffic.

* Made by Watson-Stillman Co., Roselle, N.J.

mained inside the shaft area, and a new length of pipe was welded to it.

A careful check was kept of the jacking pressures for each of the sixteen shifts. (There were two shifts during the last three working days.) The pipe's alignment, when necessary, was varied by reducing the pressure on one jack. The jacking load for the first 25 ft of pipe reached 150 tons, and for the remaining 75 ft, the loads varied between 150 and 230 tons. This was well within the 400-ton capacity of the two jacks.

Time of jacking shutdown, based on the number of working shifts per day and shutdown for weekends, had a marked effect on starting pressures and total force each time work was resumed. The greater the lapse of time, the greater the starting force required. For two-shift operations (no work for 8 hr), the starting pressures increased approximately 4 per cent on the average, a negligible amount. For one shift (no work for 16 hr), the starting pressures increased approximately 28 per cent. For weekend shutdown (no work for 64 hr), the average increase was 41 per cent.

These percentages, based on a limited number of readings, will, of course, vary for different conditions and soils. There is a need for further field study of starting pressures to obtain more accurate data. It is obvious, however, that work of this type should be carried out on at least a two-shift basis, to eliminate any appreciable increase in starting pressures. A three-shift schedule may be desirable under certain conditions.

Installation of Water Pipe

When the full length of the steel pipe had been installed, a 48-in. ID reinforced-concrete water pipe, in 16-ft lengths, was placed inside it by jacking. A concrete cradle was first installed by the general contractor. The steel interlining of the 48-in. concrete pipe was welded together at the joints as each 16-ft section was placed in the shaft. Two 30-ton jacks with 5-ft strokes were substituted for the original jacks in order to speed up this operation.

Installing the water pipe required a much larger jacking force than anticipated. Approximately the same force as the weight of the pipe (37 tons) was exerted before the entire length was pushed through to the opposite end. The coefficient of friction was slightly over 1.0.

Grouting Between Pipes

The third phase of the job required the grouting of the concrete pipe inside the steel shell. Bulkheads were built at the exposed ends of the steel pipe on each side of the road to close off the 4.5-in. annular space between the concrete and steel lines. A 10-ft high, 1.5-in. vent pipe was attached to each bulkhead to insure that no air was entrained in the annular space and that the space was completely filled. Grout was pumped into the 102-ft length through a 2-in. flush-joint pipe * that was previously supported at the top with **J** hooks within the steel pipe. As grouting proceeded, the 2-in. pipe was withdrawn.

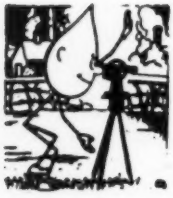
It took 8 hr to fill the annular space between the two pipes with 28 cu yd of 1:2 grout. With a high-speed

* AX pipe, in the standards of the Diamond Core Drill Manufacturers Association, New York, N.Y.

(2,000-rpm) grout mixer,* the grout was simultaneously mixed and pumped directly into the void at a pressure of 20–30 psi. The grout mixer, because of its high-speed high-shear mixing action, produced a grout with a low ratio of water to cement (0.53), but with high fluidity and pumpability. It also had the unique property, by nature of the centrifugal pump action of the mixing elements, of serving as a low-pressure pump as well as a mixer. Thus, less manpower and plant investment were used, for no separate pieces of pumping equipment were required. Each batch of grout mixed consisted of three bags of cement, 6 cu ft of sand, and 18 gal of water.

The jacking of the steel pipe was completed in less than 3 weeks; the entire project, in 7 weeks. Work in the field was carried on by a ten-man crew.

* Colcrete DD-8 colloidal grout mixer, manufactured in England and assembled in the United States, with an American-made motor, by Colcrete Structures, an affiliate of Spencer, White & Prentice, New York, N.Y.



Value and Use of System Records in Long-Range Planning

Russell G. Kincaid

A paper presented on Sep. 29, 1959, at the Missouri Section Meeting, Kansas City, Mo., by Russell G. Kincaid, Partner, Burns & McDonnell Eng. Co., Kansas City, Mo.

CURRENT literature on water utility needs puts much emphasis on the subject of system deficiencies. A recent USPHS survey of utilities serving populations of 25,000 or more indicates that these deficiencies have reached very serious proportions. The percentages of systems showing deficiencies in the principal categories are [1]: supply 20 per cent; transmission capacity, 40 per cent; treatment plant capacity, 40 per cent; pumping capacity, 33 per cent; distribution system, 57 per cent, including ground level storage facilities, 29 per cent, and elevated storage, 43 per cent.

Water utilities often develop deficiencies because of lack of public support. Too frequently, local authorities fail to take action in time to prevent serious shortages. The only positive solution to this dilemma is to provide professional management for water utilities. This subject is effectively discussed in a recent article by Raymond J. Faust.[2]

Supply and Treatment Records

The records that most utilities keep on supply and treatment facilities are, when properly interpreted, usually adequate for planning purposes. In the long-range planning of supply and treatment facilities, the most useful records are those that show the historical trend in demand. The average annual day, the maximum day, and maximum hour for about 20 years should be known. Records of hourly pumpage and consumption for several maximum days each year should be studied and summarized. Sufficient record data should be studied annually to develop daily-demand curves for maximum days and maximum weeks. The weekly-demand curves should sometimes be developed for: filtration rates, low-service pumping, high-service pumping, and high-service consumption.

Other records that are useful in the long-range planning of supply and treatment facilities are those on adequacy of supply, quality of supply, demand categories (industrial, commercial, manufacturing, and domestic), population served, per capita consumption, number of services, water unaccounted for, rainfall and temperature data.

In order to anticipate what improvements will be necessary in the supply facilities of any given utility, reliable information must be available on separate plant elements. This includes information on the overall hydraulic limitations of the plant and each of its elements; the safe rated capacity of all

basins and filter units; the delivery, under present and future head conditions, of all low- and high-service pumps, operating both individually and in various combinations (including the largest unit out of service); the capacity of all basin equipment and chemical-handling and -feeding equipment. In short, sufficient data must be available to insure that the weakest link in the plant chain can be strengthened *before* any break actually develops.

ately to its ultimate capacity of 140 mgd. Furthermore, an engineering study was authorized for the purpose of developing a master plan of system improvements beyond a capacity of 140 mgd.

A special study of the abnormal increase in maximum day demands was made. This study revealed that the rapidly increasing use of nonconserving type air conditioner was causing a runaway demand simultaneously with

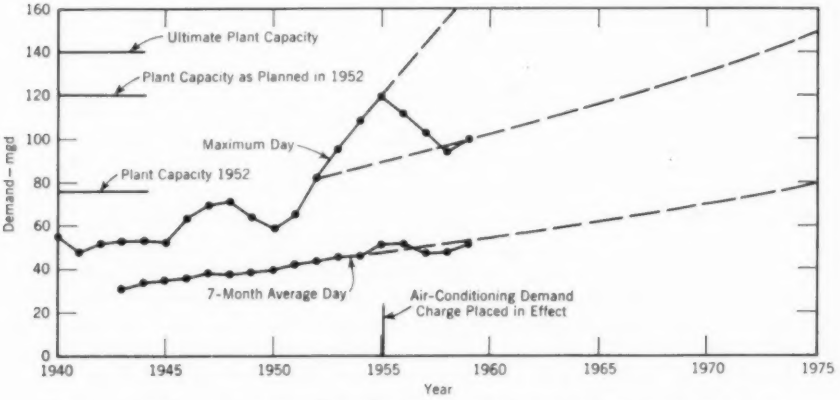


Fig. 1. Effect of Air-Conditioning Demand Charge on Maximum-Day and 7-Month Average-Day Trends, Omaha, Neb.

The dashed portions of the curves represent trends predicted as a result of the imposition of the air-conditioning demand rate after 1956.

Air-Conditioning Demands at Omaha

Figure 1 shows the trend of the maximum day system demand at Omaha, Neb., for the period 1939 to 1959. In 1952 an improvement program designed to increase the plant capacity from about 75 mgd to 120 mgd was begun. The abnormally high demands experienced in 1953, 1954, and 1955 resulted in an accelerated plan of improvements. It was decided that the plant should be brought up immedi-

normal summer peaks. It was concluded that the growth trend of city demand could be represented best by considering the 7 months from Oct. 1 to May 1. This 7-month average-day trend is shown graphically in Fig. 1.

In 1954 utility officials were advised that unless steps were taken to combat the loads imposed by air-conditioning equipment, the ultimate capacity of the present plant would be exceeded before a new source of supply could possibly be developed. Following the increas-

ingly high demands experienced during the summer of 1955 it was decided to place a demand charge of $36 per ton per year on all nonconserving air-conditioning installations. This demand charge became effective Jan. 1, 1956, and its immediate effect on the maximum-day demand can be seen in Fig. 1. The change in trends is partly because the dry years, 1953, 1954, and 1955, were followed by abnormally wet years. The maximum-day demand rose from 80 mgd in 1952 to 120 mgd in 3 years and then dropped back to about 94 mgd by 1958. The 1959 maximum day is on the predicted maximum-day trend line.

The Omaha experience shows that the load imposed by nonconserving air conditioners is probably the most difficult type of load to be dealt with in a water system, for, as soon as rates are adjusted to be consistent with the cost of providing service, the load may disappear almost completely, owing to the installation of water-conserving equipment. The net result of this experience at Omaha was that extra system capacity was provided several years before it would otherwise have been necessary. The 1952 plan of increasing the plant capacity to 120 mgd would have been adequate until about 1967, had it not been for the temporarily high air-conditioning load.

Figure 2 has been included to show hourly system demand at Omaha during the 1955 maximum day. It should be noted that the peak-hour rate was 157.3 mgd, whereas the total consumption for the day was 119.3 mil gal. Because of adequate storage at the plant and on the distribution system, plant capacity is determined by maximum-day demands. High-service pumping capacity is set 10 per cent higher than maximum-day demands. The 1940 maximum-day curve is included to show the change in the shape of the maximum-day load curves representing the effect of the nonconserving air-conditioning loads experienced in 1955.

System Analysis

The development of the Hardy Cross system 20 years ago has gradually

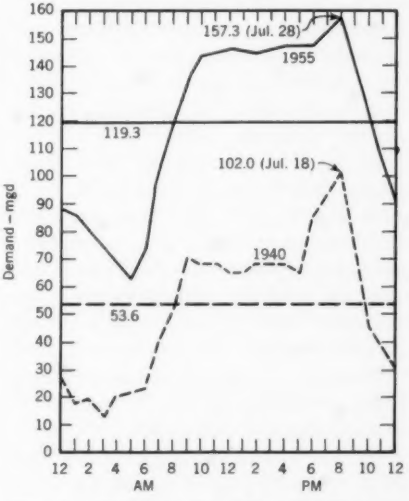


Fig. 2. Maximum Day Demand Curves for Omaha, Neb., in 1940 and in 1955

Comparison of the 1940 curve with the 1955 curve shows the effect of nonconserving air-conditioning equipment (used primarily during the hot, daylight hours).

brought distribution system analysis from the rule-of-thumb category into the realm of sound engineering principles. Recently the McIlroy analyzer and the electronic computer have replaced older, tedious methods of analysis. Basic data must, however, still be compiled before any method of

analysis can be applied. It is therefore essential that complete and reliable records be available.

Input Data

The maximum-day conditions that are usually analyzed are maximum-hour demand and replenishing storage. It is necessary, in the study of maximum days, to have hourly data on all sources of supply to the distribution system, including high-service pumps, gravity supply, booster stations, and ground and elevated storage. Data on the source of supply are usually available in the form of 24-hr flowmeter charts. Storage inflow and outflow rates may be determined from flowmeter records or level records. The level records may be either on charts or tabulated from data on storage capacity at any water level. Hourly records of discharge rates, hours operated, suction pressure, and discharge pressure should be available for all high-service pumps and booster pumps. The current head-capacity curves, of course, should be available.

Figure 3 is an example of a distribution system network condensed to simple form. The only sources of input during the maximum hour are the supply works and storage. The rates from these sources are obtained, of course, by analyzing maximum-day load curves. At the source, hourly pumpage rates are required. Hourly water level records for distribution storage reservoirs or elevated tanks are essential if backflow rates during peak hours are to be known. Although these are the most fundamental records needed for a distribution system, it often happens that no information is available on storage operation. This means that the shape of the maximum-day demand curve can only be guessed, no actual record being available on maximum-hour demands.

Pipe Records

Before the distribution of flow in the network can be computed, other basic data are necessary. The size, age, and length of each pipe section in the skeleton network must be known. These data are usually, but not always, obtainable from records. Complete records should be kept on the installation of all new distribution piping and should include the date of installation, size and type of pipe, type of pipe lin-

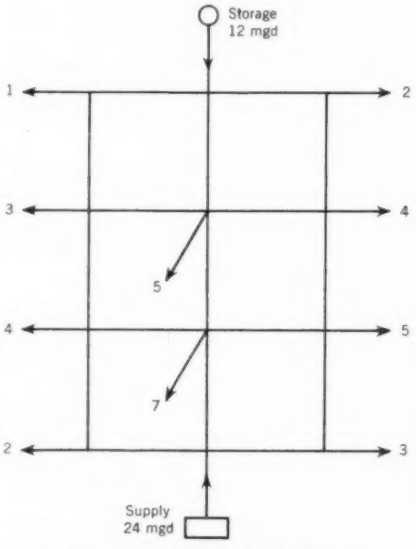


Fig. 3. Condensed Distribution System Network

The simplified network shown illustrates the necessary basic data for an analysis. Input from supply and storage give, in this example, a maximum-hour rate of 36 mgd.

ing, if any, and length of pipe laid. Records should be kept of cleaning and relining of old mains. Details of connections to existing distribution piping and of valve locations are also essential.

Water Use Data

In addition to data on network piping, it is necessary to know the pattern of water use throughout the system. Except for special surveys, the only sources of information available on the distribution of water usage throughout the system are meter reading records. Usually, this information is used only by the commercial department for computing water bills. When the Hardy Cross system of analysis was first used, maps of meter reader routes often were not available. Maps could be developed only by the laborious process of referring to individual meter books for routing locations. Because of this situation, the author suggested, as early as 1944,[3] that the distribution of water usage throughout the system should be checked annually. The meter book district usually is the most convenient unit of area for this purpose. If meter reader routes are made to conform to the boundaries of established census tracts, it is possible to follow the trend of population, water usage and per capita consumption in each of the population subdivisions of the city. The use of this principle is very ably described by Graeser,[4] in connection with the use of the Hardy Cross analysis in Dallas:

> It is planned to use the meter reading data according to read districts and census tracts as time goes along, and to use the same load centers as those of the 1980 master plan to determine whether or not pressures will be low in a certain area and what areas should be given priority in construction as growth continues.

The punch card records now in use at Dallas provide a wide variety of information very useful in long-range planning of system improvements as well as in making rate studies. Many other valuable records are also available in addition to the primary functions of billing and accounting operations. Before discussing further the basic data needed for distribution system analysis, certain information relating to census tracts and political wards as population subdivisions will be presented.

Census Tracts

Census tracts as established by the US Bureau of Census are small areas having a population of 3,000–6,000. Certain large cities and sometimes their adjacent areas have been subdivided into tracts. These tracts are intended to remain unchanged from census to census.

There are 12,633 tracts in 69 tracted areas for which 1950 census data are available. Eight tracted areas were originally formed in 1910 and have increased in number since that time to 69.

Tracts are created for statistical and local administrative purposes. They are established with approximate uniformity in population, and some consideration is given to uniformity of size, with due regard for natural features. Tracts may form subdivisions of wards in areas where ward lines are infrequently changed, but they are usually laid out without regard to ward boundaries.

Kansas City and St. Louis were the only Missouri cities subdivided into census tracts for the 1950 census. The

number of tracts in these cities in 1950 were: Kansas City, 99 (adjacent areas, 9); and St. Louis, 128 (adjacent areas, 119).

Other cities are now in the process of being divided into census tracts for the 1960 census. For example, Springfield, Mo., is now divided into 25 tracts.

In Kansas City, many of the tract boundaries conform to section lines. Tracts may, for example, be ¼, ½, or 1 sq mi in area, depending upon their locations and populations.

Census data on population by political wards are available in all cities. Where census tracts have not been established, wards may be adopted as subdivisions of the water distribution system, but ward boundaries do change frequently, and it is therefore advisable to establish more permanent subdivisions.

TABLE 1

Tract Data Compared to Data for an Entire City

Item	Tract	Entire City
Population	4,000	100,000
Consumption—*gpcd*	125	120
Average day—*mgd*	0.50	12.00
Maximum day—*mgd*	0.80	20.00
Maximum hour—*mgd*	1.50	36.00
Fire flow—*mgd*		12.96
Maximum day plus fire flow—*mgd*		32.96

Suggested Subdivision

An effective analysis method is to subdivide a city of a given population into tracts of suitable size and population for distribution system records. In this example, the map of a city of 100,000 has been divided into 24 tracts. The principal records and information desired for each of these tracts are indicated in Table 1. Other data are also important, such as fire flow requirements and classification of consumption as to domestic, commercial, manufacturing, and industrial use. These records should be kept current, especially for rapidly expanding areas. Distribution system analysis becomes quite simple if such data are available. As more and more analyses of assumed future conditions are made and compared with actual conditions, the degree of accuracy of the analysis improves. The distribution system becomes a combination of unit areas of suitable size in which reliable water use data are available. Growth trends can be detected readily and the missing links in a master plan can be filled in in proper sequence. An orderly development of transmission and feeder mains is thus accomplished.

Other Useful Records

In addition to the records previously discussed, several other types of distribution information are desirable. Recording pressure gages should be installed on principal feeder mains throughout the system, and particular attention should be given to critical pressure points on high ground. Permanent metering stations or pitot meter taps should be established on all principal feeder mains. Flow tests should be conducted on maximum days. Information on fire flow requirements throughout the system should always be up to date.

Aerial maps and topographic maps should be available as well as information on the elevations of all pump stations and storage facilities. Information should be readily available on possible sites for future reservoirs, ele-

vated tanks, booster stations, and other improvements.

A list of largest consumers showing location and water consumption trends should be kept current. The probable location of new industries should be studied. Records should be kept on the extent and location of building permits.

Central Control

The establishment of a point of central control of distribution system operations is highly desirable. In such a control center, information can be made constantly available on:

1. Pressure and pumping rate indications at all stations and at critical points throughout the system

2. Reservoir levels and elevated-tank levels (Altitude valves on elevated tanks may also be remotely controlled.).

Conclusion

The orderly development of any water system depends upon effective long-range planning. A planned pattern in which the various stages of improvement fit into a master plan is essential. This, in turn, is possible only when there are adequate records, particularly distribution system records, for system planning.

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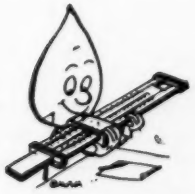

Installation and Maintenance of Distribution Facilities in Wisconsin

Panel Discussion

A panel discussion presented on Sep. 10, 1959, at the Wisconsin Section Meeting, Milwaukee, Wis.

Water Main Installations at Neenah—*Everett Westfahl*

A paper presented by Everett Westfahl (deceased), formerly Supt., Water Dept., Neenah, Wis.

IN 1942, Neenah, Wis., had 43.6 mi of water main; the largest main at that time was 14 in. Although the city did not have a large distribution system, there were so many water main leaks and breaks that in the fall and winter months it was difficult to keep them all repaired. Because of the extensive number of failures, a survey was made of the entire distribution system, including water mains and valves.

All available information was collected and a careful record made of new leaks and breaks. These data were compiled and a map made of the distribution system, locating leaks breaks, and valves. A study of the data and the map revealed that the breaks and leaks were not confined to any one area; nor were they the result of the work of any one contractor. Expansion of water main, poorly dug trenches, blocking under pipe, pipe resting on boulders, and a few instances of pipe shearing at the wall of valve manholes seemed to be the usual causes of the difficulties.

In order to repair these mains, it was necessary to shut off the water in some areas for more than six or eight city blocks. The valves in the system, like those in the systems of most older cities, were in very poor condition and far apart. It took a lot of man-hours to make repairs—which were becoming more and more costly—and because the water was often shut off, many customers complained.

New Installations

All the mistakes of the past were considered before new specifications for laying water main were prepared. In 1946, the first mains were installed under the new specification. The results since that time show that the city now has a good, practical system.

With the new specifications, there is installed a 6×15-in. solid sleeve, with 6 in. between the spigot ends of the water main, in every intersection just past the valve (Fig. 1). At each intersection, three valves in concrete-block manholes are installed. With three valves for each intersection, the water in each city block can be shut off, facilitating repair work. The water main is laid with a minimum 6-ft cover, measured from the existing or proposed street grade. It is specified that all trenches be dug level, with no dips or sags, and that beli holes do

not exceed 18 in. in length and 8 in. in depth. When a joint is made and tested, the area around the bell must be filled and tamped with crushed stone or gravel before covering. Blocking of any kind under water main or valves is not permitted.

If the bottom of the excavation is not dug to true grade, the contractor, at his own expense, must bring the excavation to true grade with crushed stone. All water main must be bedded on 4 in. of crushed stone or crushed gravel. After the main is installed, the contractor must test it at a pressure of 100 psi for 24 hr. After the 24-hr test, if all joints are watertight, the contractor backfills with crushed stone or crushed gravel up to and covering the top of the water main.

As mentioned before, concrete-block manholes are used for all valves (Fig. 2). The manhole does not have a concrete bottom, but is built on 8 in. of crushed stone. At first, the main

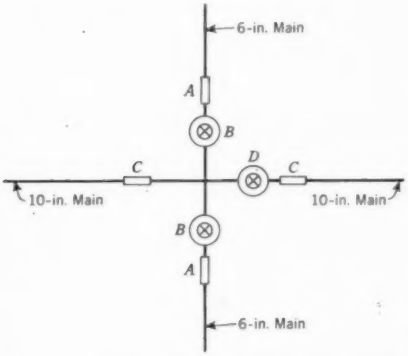

Fig. 1. Typical Water Main Intersection at Neenah

The lettered portions of the diagram represent the following: A, *6×15-in. solid sleeve;* B, *6-in. valve and manhole;* C, *10×18-in. solid sleeve; and* D, *10-in. valve and manhole.*

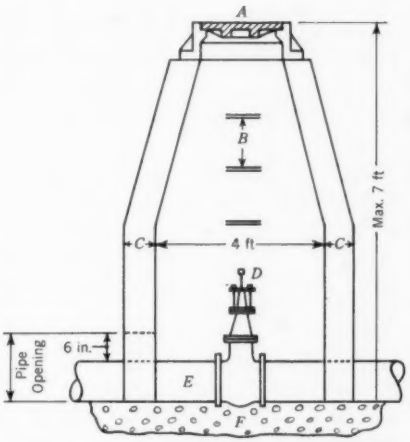

Fig. 2. Typical Water Valve Manhole at Neenah

The lettered portions of the diagram represent the following: A, *lid;* B, *ladder used in deep manholes;* C, *6-in. concrete block, brick, or precast concrete walls;* D, *valve;* E, *main; and* F, *8-in. layer of crushed rock.*

was plastered solidly into the manhole wall, but after instances of pipe shearing at the wall were discovered, an opening was left for the pipe to pass through. With this opening, which extends to 6 in. above the top of the pipe, all shearing failures at the manhole have been eliminated.

Results

Today the city has 60.4 mi of water main, ranging in size from 6 in. to 16 in. It is believed that the solid sleeves for expansion and the crushed stone under and over the water main have solved the difficulties with main breaks. At present, the distribution system is operated with one foreman and three laborers, who make all water taps, install water services up to the meter, set the meters, and make minor repairs.

Valve and Hydrant Maintenance at La Crosse

—*Clyde Collins*—

A paper presented by Clyde Collins, Supervisor, Distribution System, Water Dept., La Crosse, Wis.

The distribution system in La Crosse, Wis., consists of 137 mi of water main, ranging in size from 6 in. to 24 in. To date, more than 1,407 main valves and 1,040 hydrants have been installed.

When the need for valve repair was first noticed, after routine shutoff of a water main, a maintenance schedule was ordered by the city's director of public works, and a survey of the distribution system had to be made. Because of the concern about valve failure and the consequent unchecked flow of water from broken mains, the repair of valves was to be the first task. There was little or no information on the valves, with the exception of their approximate location as shown on the distribution map. It was not known just how accurate the map was, but because it was the only guide available, it had to be followed. One street at a time was surveyed and the valves shown on the map located. When only one or two valves were shown to be located at an intersection, an attempt was made to find others. Sometimes one or two more valves were found. Information was recorded about valve size, location, type of housing (manhole or valve box), direction for closing (clockwise or counterclockwise), and the number of turns required for closing.

Valve Repairs

During the survey, some valves were found to be covered with concrete, to have broken stems, to be without stems and gates, and to have inoperative wedge gates due to corrosion. Some valve boxes were broken off at 2 ft and 3 ft below the street grade. By the time six streets had been surveyed, it became necessary to stop the survey and start the repair work. Often the repair work had to be done at night, because businesses were in the shutoff area. Once out of the business district and into the residential district, repair crews were able to work during the day. Customers were notified, a day in advance, of the approximate number of hours that the water would be shut off. With most valve repairs, it was necessary to make two shutoffs, once when the broken parts were removed, and again when the new parts were ready to be installed.

When the survey and repairs were completed, all data on valves were put on index cards, including diagrams showing valve location in the intersection. Then the distribution map was corrected. Today, each maintenance truck has a book with an accurate map and all the information on valves. The distribution system is now well under control; valves are inspected and operated at least once a year and repairs made immediately.

Hydrant Repairs

After the valve survey had been completed, the hydrants were inspected in a similar manner. Data recorded on index cards indicated the hydrant location, size, lead from the main, pres-

sure, number of nozzles, control valve, and the date of installation. When the file was complete, a maintenance and inspection form was drawn up, listing all work that should be done on each hydrant, such as repair of stuffing-box leaks, greasing, checking of nozzle threads, and painting. A crew of two men operated and inspected each hydrant and did the work listed on the form, recording any defect in the operations of the hydrant that could not be repaired during the inspection. A followup maintenance crew made the major repairs. The principal problem is that many of the hydrants do not drain; because of this, it will be necessary to inspect each hydrant twice a week during the winter months. With so many different kinds of hydrants in the city, each hydrant requiring a different method of repair, this maintenance problem is going to be a very difficult one. The city is attempting to eliminate the problem of draining by cleaning the drains, packing gravel around the bottom, and covering the gravel with a heavy roofing paper. All new hydrants in La Crosse are installed this way and, as far as draining is concerned, have given very little or no trouble. It is hoped that the hydrants will soon be in such good condition that it will not be necessary to have them inspected during the winter months, and that one man will be able to maintain them during the entire year.

Thawing of Water Services at Marinette

Robert J. Poss

A paper presented by Robert J. Poss, City Engr., Marinette, Wis.

During the winter of 1958–59, Marinette, Wis., had 46 days of zero or below-zero temperatures. These conditions, coupled with an almost total absence of snow until well into the cold period, resulted in frost penetrations as deep as 7 ft. Consequently, more than 700 service pipes, two mains, and a number of hydrants froze.

The thawing of mains and hydrants was not a particularly difficult problem, but the thawing of more than 700 frozen service lines was a major task. Maintenance crews worked 16–18 hr every day of the week, and were diverted from thawing operations only for emergencies. Late in January 1959, it became evident that the city could not cope with the ever increasing number of freeze-ups. A local welding contractor, who had a jeep-welder of high capacity, was hired on an hourly basis and assigned to the more difficult or more persistent freeze-ups. As the number of calls increased, two other persons who owned welders assisted the city on an independent basis, charging the property owner the same fee as the city charged—$10. The city's arrangement with the independent workers prescribed that they be responsible for any damage to property resulting from the thawing.

When a service was thawed, the customer was asked to let the water run, with the understanding that an adjustment would be made on his water bill. Most customers followed instructions; others thought it wasteful to let the water run, turned it off, and soon were forced to request another thawing job.

The city used more water during this period than it ever had. A new filter plant had just been completed and was being tested when it became evident that the old plant could no longer handle the pumpage needed. It then became necessary to use the facilities of the new plant in addition. This was done without loss of volume or impairment of water quality. The city's average pumpage during the winter is approximately 1.7 mgd; during that winter, it was almost 3 mgd.

Thawing Technique

A portable welder and a current of about 175 amp are used in thawing operations. On some persistent freeze-ups, more amperage may be required. The meter and ground wire are disconnected from the house lines before the welder is hooked up. This prevents the current from entering house circuits and doing damage to house wiring or appliances.

The city's older mains are lead jointed. From 1946 to 1957, mechanical-joint pipe with lead-tipped gaskets was used. Hooking to the hydrant nearly always gave a good connection. In 1958, slip-joint pipe was tried, but no provision was made for a metallic connection between pipes. This ommission resulted in difficulties. In some places, it was necessary to dig down to the shutoff; in other places, if houses were directly across the street from each other and were hooked up to the same length of main, thawing was done from the respective curb stops or from service pipes within the houses. This procedure has been changed. The pipeline inspector is now equipped with lead wedges and a circuit tester. The wedges are put in place as the pipe is laid, and then, at the end of a day's backfilling, the circuit is tested to see that there is conductivity. If there is no conductivity, the contractor digs to the joints until the break in the circuit is located.

Liability

In the event of property damage resulting from thawing operations, the utility pays for the damage. Inasmuch as the utility charges for its services, its functions must be considered proprietary and not governmental. There have been instances when the utility believed that claims were made for damages not due to thawing, but because this could not be proved, the utility's insurance carrier paid.

Recommendations

From the studies made at Marinette, the following procedures prior to thawing operations are recommended:

1. Disconnect the water line at the water meter.
2. Remove the ground clamp from the water pipe.
3. Close the main house switch or disconnect the neutral wire at the meter to prevent a stray current from getting into the neutral wire and out into the system.

It is also emphasized that cast-iron sewers and gas lines should be laid no closer than 3 ft from the service line. On one occasion, a gas line was laid close to the water pipe; a current used for thawing passed to the gas line, melted a lead fitting at the meter, and caused a house to become filled with gas.

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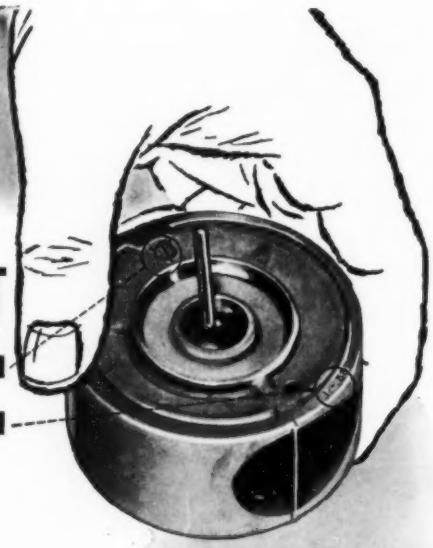

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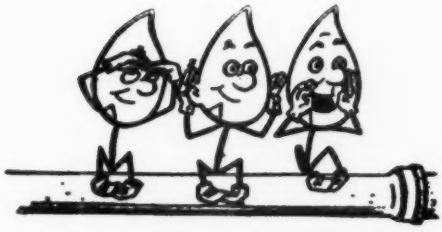

Percolation and Runoff

'The good old days' reared an ugly headline again last December, featuring a waterborne typhoid epidemic at Keene, N.H., a city of 16,000 population. Behind "the good old days," of of course, were "the good old ways," the community having depended upon slow sand filtration alone to treat a surface water supply drawn from a "protected" watershed. Actually, it took no more than a single member of a three-man logging crew to pierce the protection, he being one of those unwitting carriers who had himself never been ill with typhoid. Helped along by some heavy rains that washed them from the carrier's wastes into the reservoir, the typhoid organisms managed to penetrate the filter in large enough numbers to get thirteen cases under way before emergency chlorination could be undertaken.

Fortunately for those stricken, the organisms were attenuated and the attacks were mild. Fortunately for those not stricken, "a better new way" was available to protect them in the emergency. But it would seem most unfortunate if the incident does no more than "reinforce the attitude of water supply officials who insist that multiple use involves definite hazards." Unless residents of and tourists to New Hampshire are different from other people, regardless of protection there will be "use" of watersheds by at least occasional uninspected individuals whose organisms may not be attenuated.

The state department of health, which now encourages, but does not require, chlorination of surface water supplies, has indicated that it will conduct a thorough investigation of the Keene epidemic. Out of its findings, perhaps, will come the basis for insisting that the emphasis be changed from "protection of the watershed" to "protection of the public."

That 'sewage' stinks—terminologically, that is—is the conclusion that led to the transformation, on Jan. 1, 1960, of FSIWA—into WPCF—of Federation of Sewage & Industrial Wastes Associations into Water Pollution Control Federation. And if the change in name seems to signify a tremendous expansion in Federation activities, that, apparently, is unintended, for both WPCF President Mark Hollis and Executive Secretary Ralph Fuhrman have emphasized that the objectives and scope of the Federation will remain essentially the same. The word "sewage," it is pointed out, had become a serious handicap, particularly

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(*Continued from page 41 P&R*)

in the Federation's growing public relations activities, and a name-change committee had been hard at work for some time looking for a substitute. At FSIWA's Dallas meeting in October, the committee recommended the change, those present voted in favor of it, and it has since been confirmed by a ballot of the full membership. Now it remains for us to get accustomed to the idea of calling our sewage works friends, perhaps, "polluticians."

Concomitant changes have been the conversion of *Sewage and Industrial Wastes* magazine into the *Journal of the Water Pollution Control Federation* and the development of a new term, "wastewater," as a substitute for "sewage and industrial wastes." We haven't caught up yet with nearly all the implications of these changes, nor, we imagine, has WPCF, itself, or its widespread membership. And no doubt a considerable quantity of wastewater will run through a good many wastewater conduits and into numerous wastewater treatment plants before we do. Basically, the idea of getting rid of an undesirable term is certainly sound, and even if we mightn't have selected exactly the substitute that our FSIWA friends did, it is, after all, they and not we who had to live with "sewage."

Whatever they call themselves and the stuff with which they deal, we will continue to cooperate with them, both influentially and effluentially, as we have since they came into existence in 1928. And we are certainly happy to have them nominally happier!

Speaking terminologically, we remember our own suggestion of not so many years ago concerning the desirability of substituting the title "manager" for that of "superintendent" and "water utility" for "water works." Not only in AWWA's publications, but in the field we have since seen these preferences adopted widely, and with a great deal of favorable comment. Meanwhile, no less an organization than the Western Union Telegraph Co. has taken similar action to make its "district superintendents" "district managers." As if to prove that WPCF can mean Federation of Sewage and Industrial Wastes Associations to one person and Water Resources Council to another, we found this note in the November issue of the *Association Newsletter*:

Seeking more meaningful titles for its officers, one club affiliated with the National Management Association has substituted "manager" for "president," "superintendents" for "directors," and calls its committee chairmen "supervisors." The new titles enhance the concept of the NMA club as an executive development medium.

For Ike, incidentally, we might suggest "fore man."

It ain't what you say, apparently, it's the way how you say it—and the secret of having it heard, according to publicists, advertising experts, and popular writers, is to make it *frightening*! That's the secret of the lead article in the January issue of *Good Housekeeping*. It is entitled:

DANGER IN YOUR DRINKING WATER

It is introduced:

Every time you turn on a faucet you are committing an act of faith. You believe that the water you give your child to drink, the water with which you cook and wash your dishes is fit for human use. Don't take it for granted!

(*Continued on page 44 P&R*)

SOUTHERN HOSPITALITY—JEFFERSON PARISH STYLE

Jefferson Parish, adjacent to New Orleans and divided by the busy Mississippi River, is host to many new plants and homes. To welcome these newcomers, the parish has just added a new 500,000 gallon elevated water tank at Marrero, Louisiana. Designed, fabricated and erected by Graver, it identifies Jefferson Parish as a progressive area, ready and willing to receive new citizens. Graver can help you do precisely the same!

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(Continued from page 42 P&R)

It is advertised:

Hepatitis . . . Virus Diseases . . . epidemics from our water faucets? . . . Why should our health be imperiled by polluted water? Who is responsible for this *new* threat to millions of Americans? . . .

And inside the facts are correct as far as we've been able to check; it's just the way they are put, the way the full story isn't told, the suggestive questions that are asked. For instance, talking about the contamination of the McKenzie River above Eugene, Ore., we presume the author is correct in quoting the Lane County Medical Society as saying that it feared that even high chlorination might not destroy all the disease-producing organisms in the water supply. But, of course, he does not point out that there has been no report of disease. He notes that "experts suspect, but can't prove, that waves of vomiting, cramps, upset stomachs, which sometime whip mysteriously across a community, are waterborne." He points out that "among the most frightening of the new contaminants are radioactive wastes," and mentions a spill on the Animas River, but he doesn't mention the monitoring that is being done on many rivers or the programs conducted by the Public Health Service to train water utility men in combating these contaminants. He tells of a water supply in suburban Denver that was so bad that one housewife was quoted as saying, "I don't even bathe my children in it until I add a goodly amount of bubble bath. . . ."

But perhaps he's right! Perhaps the author, one Alvin B. Toffler, and all the high-priced advertising and public relations talent that proposes the same technique are right—that this, indeed,

(Continued on page 46 P&R)

900 feet installed in under 3 hours

. . . with time out for lunch!

As fast as trenching machines will operate . . . that's how fast it is to install AMERICAN FASTITE® Joint pipe! Numerous case histories verify this. For example, on a Southern suburban water supply system recently, 900 feet of 6-inch AMERICAN FASTITE® Enamelined pipe in 16-foot lengths was installed in under 3 hours . . . including time out for lunch. The entire operation required a minimum of labor and equipment and resulted in maximum installation economies.

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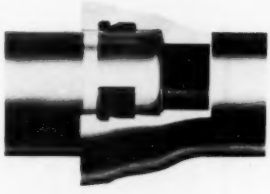

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(Continued from page 44 P&R)

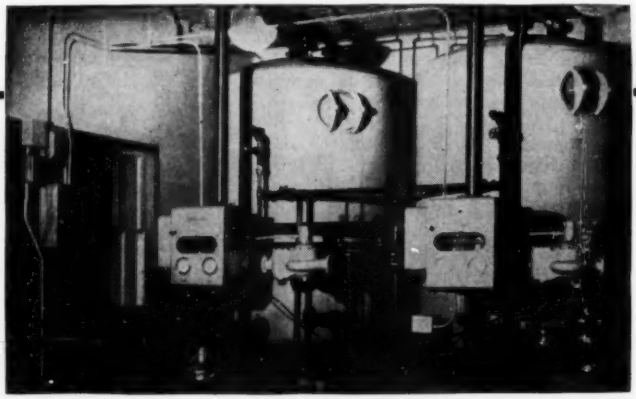
is the way to tell the public—to sell the public. And if it isn't quite honest and if it throws a shadow rather than a light on the water supply field, perhaps the end justifies the means. Perhaps people have become so accustomed to being frightened into using this toothpaste, smoking that cigaret, eating the other cereal that they can only be frightened into supporting this bond issue or that project. Perhaps. We rather doubt it, but not having the money to go about it any other way, we'll even say: Perhaps—we hope!

At any rate, "Danger in Your Drinking Water" is a story that has undoubtedly been read by a million or more people. And the end of the story may well undo some of the damage done by the rest, in telling people that they can do something about improving their water supply. They can, with your help, that is. And to make the best, rather than the worst of the article, we suggest that you read it and answer it, locally, to your women's clubs, your civic organizations, your newspaper, and whomever is willing to listen!

There's opportunity in the "Danger in Your Drinking Water"!

Marvin Lane, technical director of Graver Water Conditioning Co., New York, has been named general manager of the company, which is a division of Union Tank Car Co. Lane has been with Graver since 1949.

(Continued on page 48 P&R)



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(Continued from page 46 P&R)

The water on Venus, recently discovered by two scientists who ballooned to a height of 80,000 ft to take a relatively unhampered spectrometric look at our neighbor in space, is unlikely to have any immediate effect on the price of water here on Earth or, for that matter, on its availability. But the fact that water does probably exist there, as it has been determined to exist on Mars, brings us one step closer to the possibility that water customers also exist there. And the search for not just life, but intelligent life, on other planets is, according to Otto Struve, director of the new National Radioastronomy Observatory at Green Bank, W.Va., the basis of a current revolution in astronomy comparable in significance with the Copernican revolution of 400 years ago. Actually, the search has been tremendously extended of late through the use of improved equipment, including such tools as Dr. Struve's 85-ft radiotelescope at Green Bank. And with the completion of the 600-ft telescope now being built by the Navy and the 1,000-ft one planned for the Puerto Rican hills by Cornell University, astronomers will soon be many light years closer to those planets in our own and other galaxies where, many believe, other intelligent beings exist, some of them very probably superior to us culturally and some of them suspecting, if not definitely knowing of, our existence. On the premise that our cosmic superiors will have

(Continued on page 50 P&R)

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(Continued from page 48 P&R)

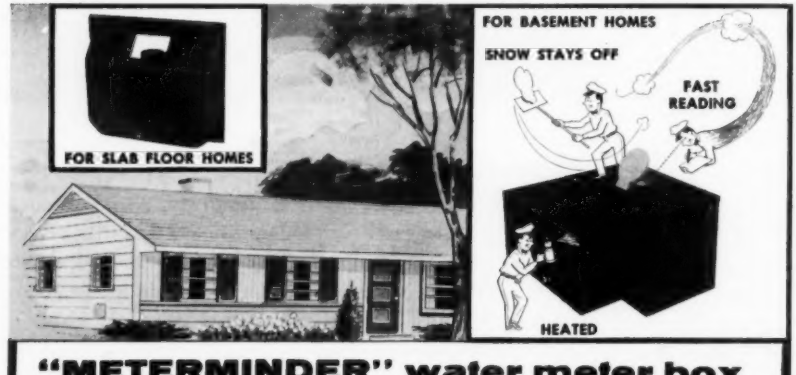
better means of communicating with us than we with them, astronomers are even keeping an ear to the sky, listening for intelligible signals from space. Just what those signals will be or say is a subject for fascinating speculation, as is the whole concept of cosmic company. But what brings us back to Earth happily is the conviction that to sustain life, planets must be watered.

Albright & Friel, Inc., consulting engineers of Philadelphia, announces the appointment to its staff of Lawson D. Matter, who retired as chief engineer of the Water Supply Section, Pennsylvania Health Dept. (Aug. P&R p. 48). Matter, who will be the firm's Harrisburg, Pa., representative, is currently AWWA director from the Pennsylvania Section.

Louis R. Howson has been named Chicago Civil Engineer of the Year by the Illinois Section of ASCE. He was cited for "excellence in his profession, leadership in the society, and for his humanities." Senior partner of Alvord, Burdick & Howson, Chicago consulting firm, Howson is a registered engineer in 30 states. He has been chairman of the AWWA Water Works Practice Committee since 1947.

Murray B. McPherson, formerly chief of the Philadelphia Water Dept. Research & Development Unit, has been appointed professor of hydraulic engineering at the University of Illinois, Urbana. The university also announced the appointment of Anthony F. Gaudy Jr. as assistant professor of sanitary engineering.

(Continued on page 52 P&R)



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(Continued from page 50 P&R)

Getting down to work was the word from Sunnyvale, Calif., and Knoxville, Tenn., last month:

At Sunnyvale the story was one of a machine that casts concrete pipe directly in the trench at substantial savings in costs. The machine, developed by No-Joint Concrete Pipe Co. of Yuba City, Calif., is essentially a round-bottomed boat, which is lowered into a trench dug to conform to the outside diameter of the pipe. Ready-mixed concrete is poured into a hopper as the machine moves forward in the trench extruding the formed pipe in its wake. Sectional aluminum forms, removed for reuse after the initial set of the concrete, support the upper half of the pipe. Unfortunately the pipe is suggested only for sewer work at the present time, but consider the possibilities of such down-to-earth techniques in water work.

And consider them is apparently what a repair crew from the Knoxville Utilities Board did, when an emergency call to repair a main break was received. Not only did the truck pull up at the scene of the trouble, it took its gear right down to the site, as it sank in 4 ft of asphalt, water, and mud. Unfortunately a round-bottomed boat or a hopper would have been better. Certainly it was no joint for a truck, the extrusion of which involved no savings in costs at all. No wake was necessary, however.

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(Continued on page 54 P&R)

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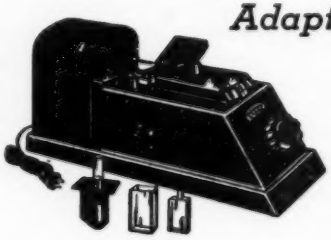

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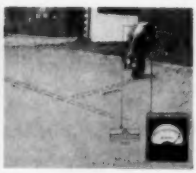

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(Continued from page 52 P&R)

Dolson

Hartung

Frank E. Dolson, vice-president in charge of distribution, St. Louis County Water Co., has been elected president and director of the utility, succeeding the late W. Victor Weir. The position also includes the presidency of the Missouri Water Co., Independence.

The company's board of directors also elected Herbert O. Hartung, vice-president and manager of production, to the newly created position of executive vice-president.

A member of the Association since 1945, Dolson received the Fuller Award in 1956. Hartung, a member of AWWA since 1938, served as director from the Missouri Section in 1955–58. He received the Fuller Award in 1954 and the Goodell Prize in 1958.

David S. Thomas has retired as district engineer, Board of Fire Underwriters of the Pacific, after 40 years of service with the agency. Having joined AWWA in 1927, he is a Life Member and a recipient of the Fuller Award (1944). He is currently the director from the Montana Section.

Philip F. Walsh has been elected to the new post of executive vice-president of Southern California Water Co., where he was vice-president in charge of operations for the past 4 years.

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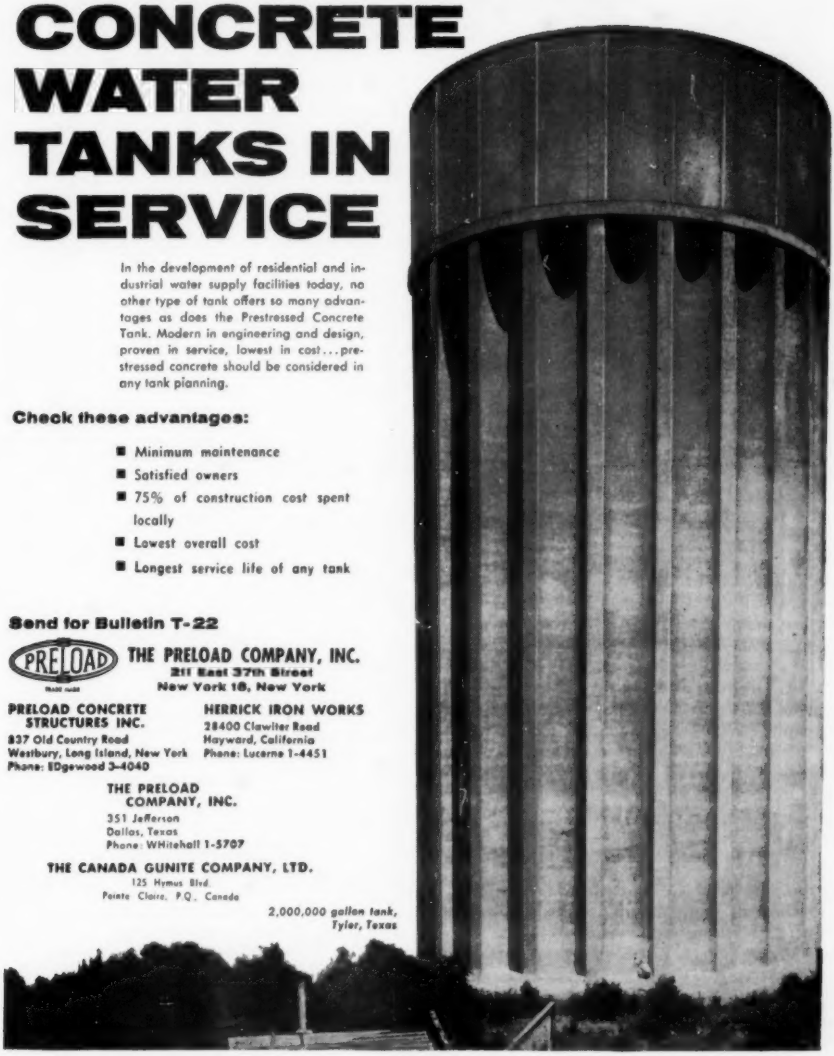

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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

LABORATORY METHODS

Colorimetric Determination of Traces of Chloride in Water. W. KEMULA; A. HULANICKI; & A. JANOWSKI. *Chem. Anal.* (Warsaw), 3:581 ('58). Colorimetric method for detg. traces of chloride in water is described. *Prepn. of a standard curve.* To solns. contg. 0, 10, 20, 40, and 60 mg Cl^- add 10 ml 0.02% diphenylcarbazone, 1 ml gum arabic, 1 ml 0.05*N* HNO_3, 2 ml 0.001*N* $Hg(NO_3)_2$, and fill with H_2O to 50 ml. Stir, leave for 20–30 min, and det. absorption on Lange's photocolorimeter with 1-cm cell and a green filter. Linear relation was found between absorption and concn. in range 0–60 mg Cl^-. *Detn. of Cl in water.* To soln. contg. 10–60 mg Cl add 10 ml of indicator (if yellow add NaOH until red) and 0.05*N* HNO_3 to yellow plus 1 ml. If soln. turns immediately red, add HNO_3 to yellow color plus 1 ml. Further procedure as above. Method was examd. by detg. Cl^- in water. Amts. of Cl^- added, those of Cl detd., coeffs. of variability, error of detn. (calcd. from 6 detns.) were: 10.0, 10.0, 9.00, —; 20, 19.0, 7.25, −5.00; 30, 30.5, 4.00, +1.7; 40.0, 39.40, 3.75, −1.50; 50.0, 49.75, 2.36, −0.50; 60 mg, 59.80 mg, 1.66, −0.33%. Mg^{++}, Na^+, Ca^{++}, Al^{+++} in concns. about 0.1 and Pb 0.003% did not affect the detn. Fe^{+++} in concns. above 0.0004 and Cu in concns. of the order of 0.00001% affected the detn. At concns. below 0.001% the effect of Cu^{++} can be eliminated by use of triethanolamine.—*CA*

Applications of Colorimetric Micro-Determination of Phosphorus in Water Treated With Polymetaphosphates. G. BENEDEN. *Bul. Centre Belge Et. Documentation Eaux* (Liege), 35:33 ('57). Author describes a spectrophotometric method for controlling addns. or detg. concns. of phosphates in water by using ammonium molybdate. Detn. is carried out in an acid medium using sodium sulfite as a reducing agent and a trace of hydroquinone to catalaze reaction. A graph shows relation between percentage absorption of color and concn. of phosphorus in mg. Several applications of this method are described.—*WPA*

Applications of Opacimetric Micro-Determination of Phosphorus to Water Treated With Polymetaphosphates. G. BENEDEN. *Bul. Centre Belge Et. Documentation Eaux* (Liege), 38:254 ('57). In the first part of this work (see above) method described for colorimetric detn. of phosphorus in water treated with polymetaphosphates was subject to interference by silica. A new method was therefore developed, based on reaction of phosphorus, in form of orthophosphate or phosphoric acid, with certain alkaloids, such as strychnine, and salts of molybdenum to form insoluble complexes in which quantity of alkaloid combined is proportional to quantity of phosphorus present. Silica does not interfere in reaction; and opalescence produced is proportional to quantity of phosphorus, following the Beer-Lambert law over a certain range of concn., and can be easily measured by means of a spectrophotometer or absorptionmeter. Prepn. of reagents and samples and procedure for detn. are described; and examples are given of application of method to assess the concn. of active constituent in a commercial polymetaphosphate and to det. phosphorus in waters treated with phosphates. Method is 10 times more sensitive than usual phosphomolybdate method, being sensitive to 1μg phosphorus. It could be used to investigate fate of hexametaphosphates added to water and changes in phosphorus in plankton and sludges. Concn. of water is not necessary for contents of hexametaphosphate of order of 1 mg/l.—*WPA*

Determination of Calcium With Eriochrome Red B as Indicator. L. JANKOVITS & L. ERDEY. *Hung. Acta Chim.*, 15:163

(Continued on page 66 P&R)

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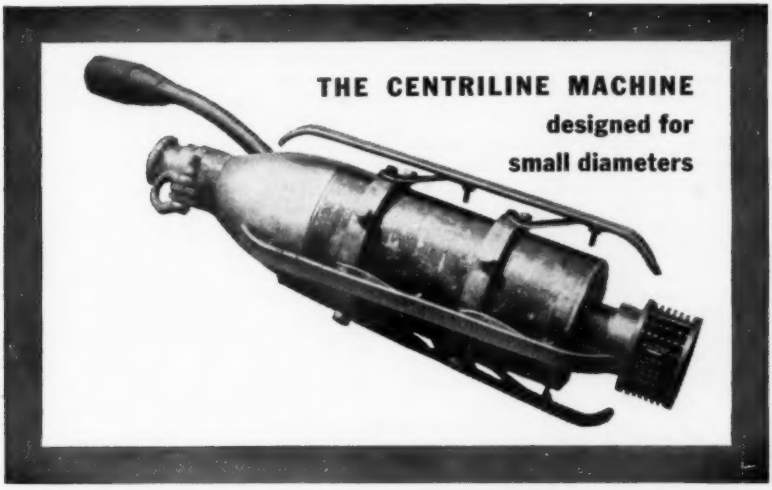

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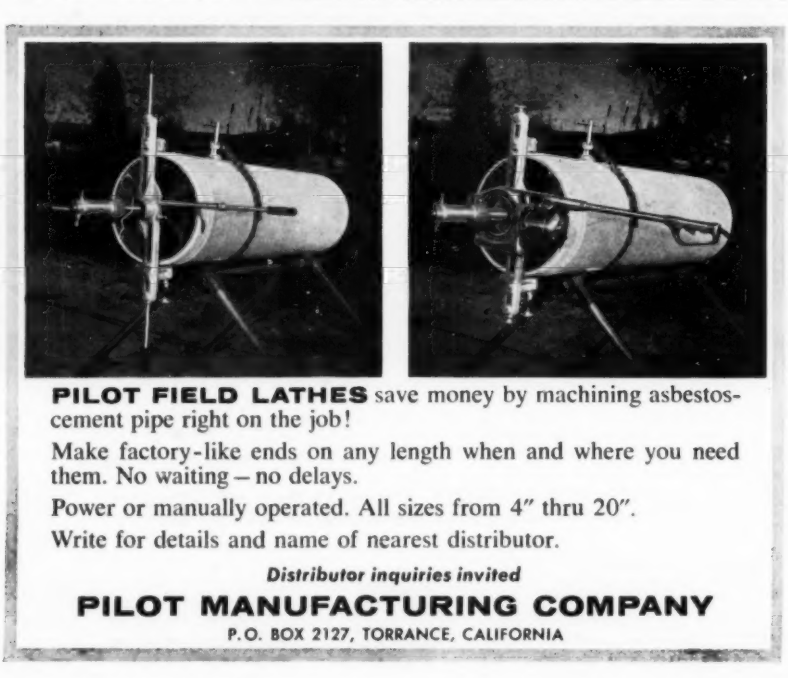
(Continued from page 64 P&R)

('58); *Analyt. Abstr.*, 5:3631 ('58). A method is described for complexometric detn. of calcium in soln. Sample is buffered at pH 9–10, and is then titrated using a 0.06% aqueous soln. of Eriochrome Red B as indicator. Endpoint is shown by a change in color from yellow to red. Interference is caused by barium, strontium, magnesium, iron, cobalt, nickel, chromium, aluminium, zinc, copper, lead, cadmium, manganese, and bismuth.—*WPA*

Simultaneous Spectrophotometric Micro-Determination of Calcium and Magnesium With Eriochrome Black T and EDTA. V. MENON & M. DAS. *Analyst*, 83:434 ('58). A method is described for individual detn. of magnesium and calcium without their preliminary sepn.; it is based on differences in stabilities of complexes of metals with Eriochrome Black T and with EDTA at same pH value. Method involves measurement of absorption at 650 mμ first of a sample to which has been added buffer soln. and a soln. of Eriochrome Black T, and secondly of a similar sample to which have been added same quantities of buffer and dye soln. and a soln. of magnesium EDTA. Absorption of corresponding blank solns. is also measured. Magnesium and calcium concns. in sample can then be calcd. from formulas. Results of analyses of synthetic mixtures of calcium and magnesium showed that method has an error of less than ±10% of individual detns. when concn. of either element, in terms of magnesium, lies between 8 and 36 μg per 50 ml of soln. Method cannot be used in presence of cations which form more stable complexes with EDTA than does magnesium; such cations must therefore be removed or suppressed by auxiliary complexing agents.—*WPA*

The Determination of Calcium in Natural Waters. J. H. CARPENTER. *Limnol. & Oceanogr.*, 2:271 ('57). Rapid and accurate procedure for detn. of calcium in natural waters involves photometric weight titration

(Continued on page 68 P&R)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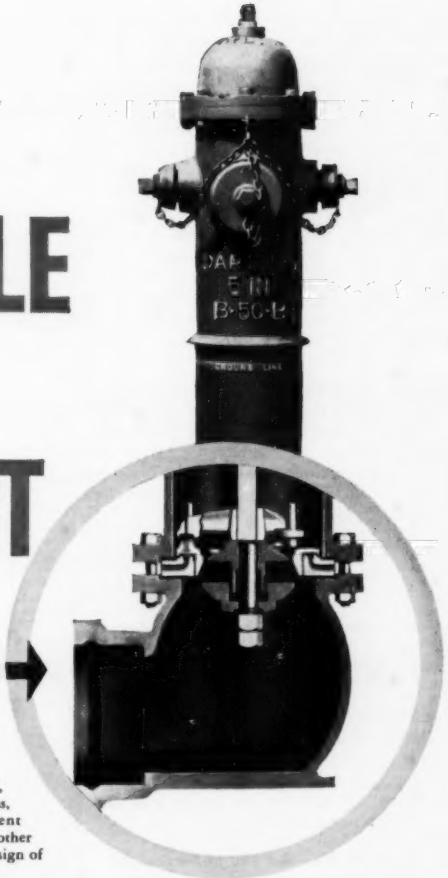

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(*Continued from page 66 P&R*)

with EDTA in presence of ammonium purpurate, after isolation of calcium by ion-exchange chromatography. Calcium is separated from magnesium and strontium by using acetylacetone as a complexing elutant. Use of method to determine calcium in sea water showed that calcium concn. was greater than would be expected from accepted calcium-chlorinity ratio for sea water. Previously calcium concn. has been detd. by triple oxalate method, in which it is necessary to make a correction for strontium, and recent work has shown that previously accepted value for strontium content was too high, and therefore corrected calcium concn. was too low.—*WPA*

Volumetric Determination of Phosphates and Metals in the Presence of Phosphates. IV. Complexometric Determination of Several Metals in the Presence of Phosphate With Eriochrome Black T as Indicator. S. S. DANIEL ET AL. *Lab. Pract. (Lond.)*, 7:15 ('58). Procedure for detn. of metals in presence of phosphate involves use of a standard zinc soln. to titrate at pH 10, using Eriochrome Black T as indicator, excess of disodium EDTA added to metal phosphate soln. This method can be used to det. zinc, cobalt, nickel, mercury, indium, magnesium, and calcium. With last 2 metals, only a small excess of phosphate will be tolerated. Alkaline earth metals, however, can be detd. in presence of phosphate by sorption of metal ion on a cation-exchange resin, washing resin until effluent is free of phosphate, and elution with dilute nitric or hydrochloric acid. Excess of eluting agent is removed by evapn. to dryness, followed by addn. of a known excess of EDTA and back-titration with a standard magnesium solution at pH 10, using Eriochrome Black T as indicator.—*WPA*

Determination of Copper in Sea Water, Silicate Rocks, and Biologic Materials. J. RILEY & P. SINHASENI. *Analyst*, 83:299 ('58). During geochemical studies of inter-

(*Continued on page 70 P&R*)

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(*Continued from page 68 P&R*)

relationships of copper in sea water, marine sediments, and marine organisms, a sensitive and specific method for detn. of copper was required. Use of 2:2′-diquinolyl as a reagent for detn. of copper in sea water was therefore investigated. This compound reacts with monovalent copper to form a strongly colored cationic complex. It was found that *n*-hexanol was a suitable solvent for extraction of this cuprous-diquinolyl complex from sea water, optimum pH value for extraction being above pH 4. Before adding reagent, sample is treated with hydroxylamine hydrochloride to convert all copper to cuprous state and the pH value is adjusted with sodium acetate buffer solution. After extraction with *n*-hexanol, copper complex is detd. spectrophotometrically. For determination of copper in sea water, 3 extractions were made and gave more than 99% recovery of added copper. Fading of color of cuprous-diquinolyl complex, which is particularly troublesome when amt. of copper is small, can be prevented by addn. of hydroquinone to *n*-hexanol extracts. Tests with various metallic ions indicated that no interference need be expected at concns. which occur in sea water. Replicate analyses on a sample of surface water from Irish Sea gave a mean value of 27.0 μg/l copper and indicated a coefficient of variation of 2.5%; similar reproducibility was obtained with water from the English Channel, which contained 19.0 μg/l copper.—*WP*

Quantitative Determination of Traces of Carbon Dioxide in Water. P. JENNINGS & E. OSBORN. *Analyst*, 82:671 ('57). A method was developed for detn. of carbon dioxide in boiler feedwater, which is more sensitive than methods previously used; attempts were made to reduce blank values to min., and size of sample was also increased to increase sensitivity. Throughout detn. sample is kept out of contact with atmospheric carbon dioxide. Carbon dioxide in sample is first aspirated for 1 hr with air free from carbon dioxide and is absorbed in 0.01*N* sodium hydroxide. Alkaline soln. formed is titrated with 0.01*N* sulfuric acid in 2 stages, using a glass electrode system

(*Continued on page 72 P&R*)

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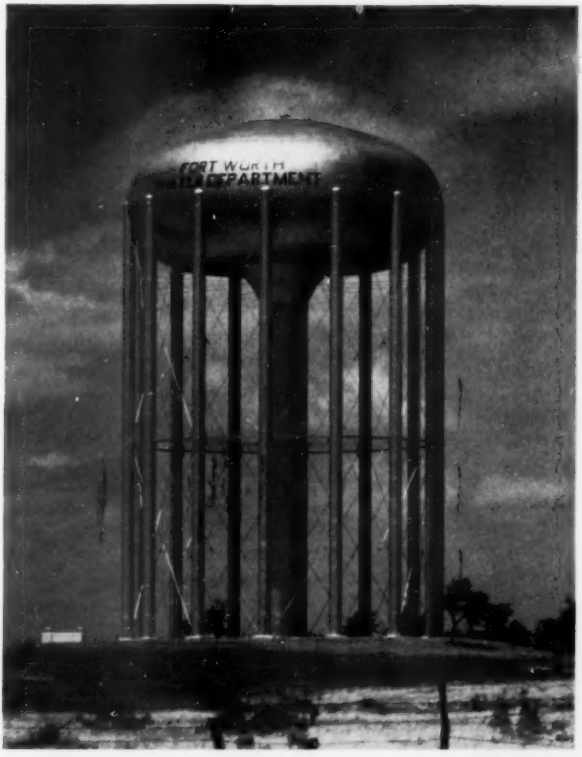

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(*Continued from page 70 P&R*)

to follow changes in pH value. Sulfuric acid is first added in a closed circuit until pH value is reduced to just below 8.3, when carbonate originally present is converted to bicarbonate. Further acid is then added, and carbon dioxide liberated from bicarbonate is removed to atmosphere by a stream of air free from carbon dioxide, until the pH value decreases to just below 5.0. Amt. of acid required between pH 8.3 and 5.0 is found from a graph, and used to calculate amt. of carbon dioxide in original sample. A similar procedure is followed for blank tests to avoid errors due to leakage into apparatus and to carbon dioxide in sodium hydroxide soln., and sample is reaspirated to avoid errors due to incomplete transfer of carbon dioxide from sample. Typical results obtained in analysis of boiler feedwaters at power stations are tabulated. In tests using standard sodium carbonate solns., satisfactory results were obtained for concns. of carbon dioxide in range 0.02–0.11 mg/l; errors occurred during tests in range 2.2–4.4 mg/l. It is concluded that carbon dioxide can be detd. in boiler feedwater with an accuracy of ±0.003 mg/l, using a 10-liter sample, provided sample does not contain more than 1 mg of carbon dioxide.—*WPA*

Salt Effect Correction in Determining Soluble Silica in Sea Water by the Silicomolybdic Acid Method. G. S. BIEN. *Anal. Chem.*, 30:1525 ('58). In detn. of a given amt. of soluble silica by silicomolybdic acid method, less color is developed in sea water than in distilled water. This effect is attributed to various salts dissolved in sea water, and varies with salt concn. Methods of correcting for effect are discussed and exptl. results are given in tables and graphs. Simplest method of correction is based on functional relation between micromolar absorbance coefficient and chlorosity. When micromolar absorbance coefficient is plotted against chlorosity, 2 linear relationships are obtained for ranges of chlorosity 0.5–5 g of chloride per liter, respectively; and from these relations, micromolar absorbance coefficient for required chlorosity can be calcd. Using Beer's law, silica content of soln. can

(*Continued on page 74 P&R*)

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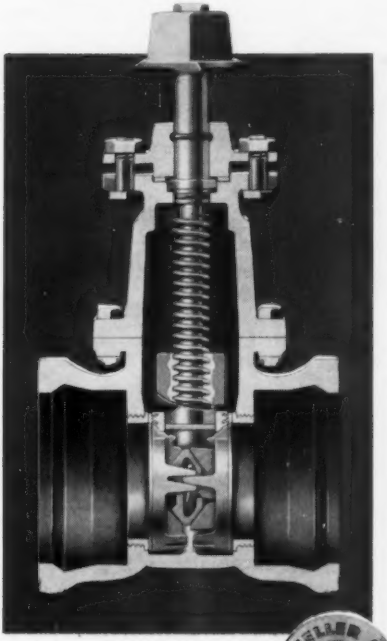

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(Continued from page 72 P&R)

then be calcd. directly from measurements of absorbance of silicomolybdate color at 360 mμ.

Determination of the Alkalinity and Borate Concentration of Sea Water. J. GAST & T. THOMPSON. *Anal. Chem.*, 30:1549 ('58). Significance of boron-chlorinity ratio of natural waters is discussed; to enable more careful study of variations in this ratio, the mannitol method for detg. borates has been improved, by replacing colored indicators by a continuously recording pH meter, using 7 g of mannitol, performing titrations in an atmosphere of nitrogen free from carbon dioxide, and controlling temp. of sodium hydroxide soln. used for titration within ±2°C. Abnormally low values for the boron-chlorinity ratio obtained in lab. tests on aquarium water were attributed to reaction of some of boric acid with polyhydroxy organic substances in water. It was shown that total borate boron in water can be detd. if polyhydroxy organic compounds are first oxidized by potassium permanganate. Details are given of procedure for detg. alkalinity and free borate boron on one sample of sea water, and tot. borate on a second sample. Difference between results for tot. borate and free borate can be used as a measure of amt. of borate combined with organic matter in water. Method was used to obtain information on relations between chlorinity, alkalinity, borates, and polyhydroxy organic matter in sea water from stations in northeast Pacific Ocean and in an estuary of Puget Sound; results are tabulated.—*WPA*

New Spot Test for Nitrate Ion. B. COLDWELL & S. MCLEAN. *Canad. J. Chem.*, 36: 652 ('58). A spot test for nitrate ion has been developed, based on yellow product formed when diphenylamine and nitrate, spotted on filter paper, are exposed to short-wave ultraviolet radiations. Yellow color is probably caused by products formed during photochem. reaction. Reaction appears to be specific and relatively sensitive, making it possible to detect 1 μg of nitrate in 0.01 ml

(Continued on page 76 P&R)

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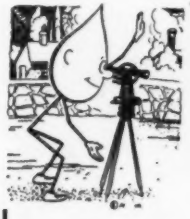

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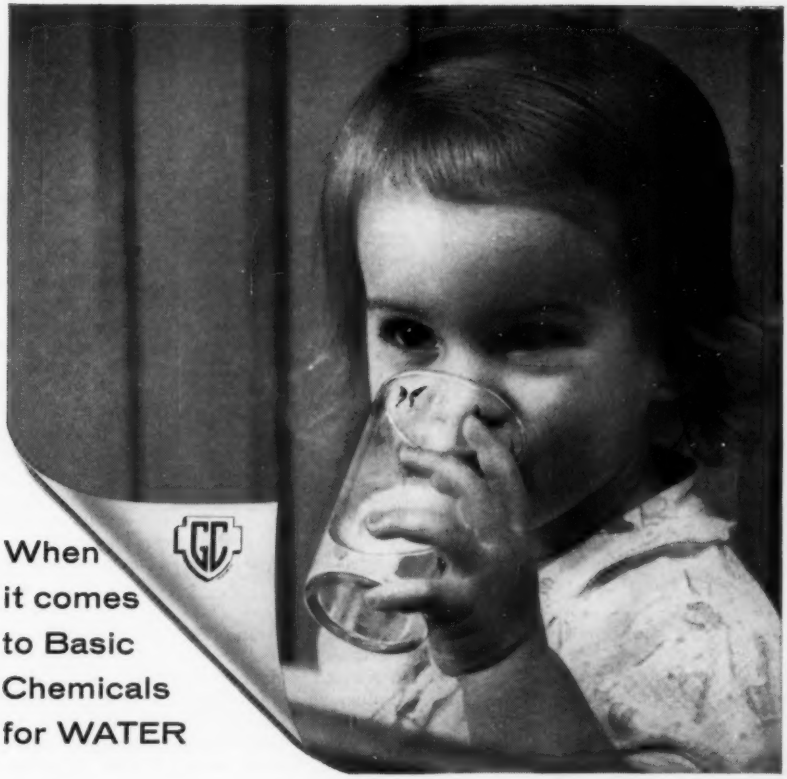

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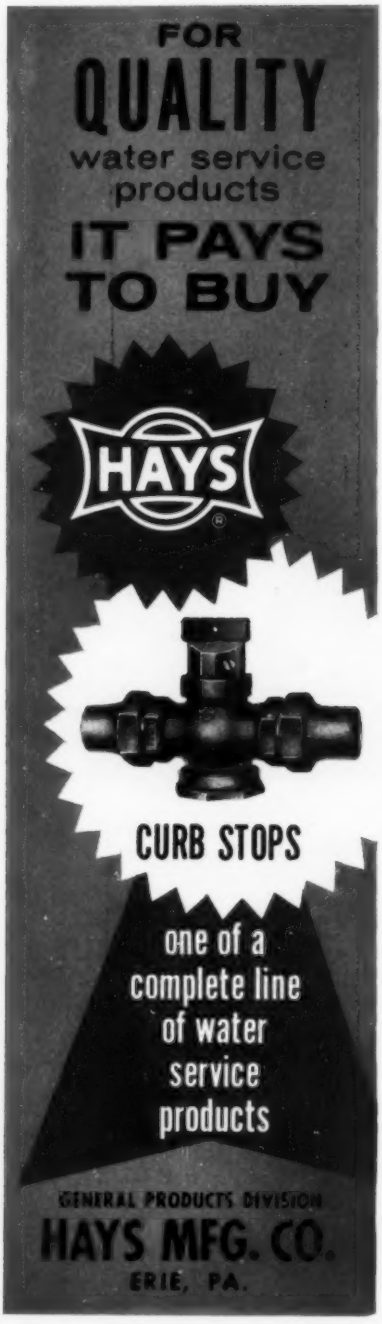

(*Continued from page 74 P&R*)

of soln. Only ions which cause serious interference are nitrite, bromate, iodate, and vanadate, and even in these 5 μg of nitrate ion can be detected in presence of 3–10 times this quantity of interfering ion.—*WPA*

Studies on the Analytical Method of Ammonia and Organic Nitrogen in Sea Water. C. MATSIDAIRA & T. TSUDA. *Tohoku J. Agric. Res.* (Jap.), 8:1:37 ('57); *Biol. Abstr.*, 32:3033 ('58). Procedure is described for detn. of ammonia and organic nitrogen in sea water. To determine ammonia, sodium bromate is added to sample and is partly reduced; excess potassium iodide and a known amt. of starch soln. are then added, and amt. of iodine released by action of remaining bromate is detd. by measuring intensity of blue color. From this, original concn. of ammonia is calcd. Results are affected by presence of amino nitrogen, but not by nitrite. In detn. of organic nitrogen, material is digested and neutralized as in Kjeldahl method, and ammonia is then detd. as before.—*WPA*

Rapid Screening Test for Cadmium. M. LIEBER. *Wtr. & Sew. Wks.*, 105:374 ('58). A rapid method for detecting presence of cadmium in potable water in field is described. Color produced by addn. of dithizone to sample is compared with temporary standard solns. containing known concns. of cadmium. Presence of hexavalent chromium does not interfere with test but nickel and copper will darken color, and these samples will require more detailed chem. analysis.—*WPA*

Determination of Low Concentrations of Sulfate in Rain and Other Waters. J. K. JOHANNESSON. *N.Z.J. Sci. Technol.* (New Zealand), 1:423 ('58). A method has been developed for detn. of sulfate in concns. less than 10 ppm in natural waters. Method is based on fact that when a soln. contg. only sulfuric and hydrochloric acids, such as is obtained by treatment of a natural water with a cation-exchange resin, is treated with mercuric oxide, mercuric chloride and mercuric sulfate are formed. The conductivity of resulting soln. is almost entirely due to sulfate, and only a small correction is necessary for chloride. Detailed procedure is given.—*WPA*

(*Continued on page 78 P&R*)

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(*Continued from page 76 P&R*)

Field Method for Determining Potassium in Natural Waters. A. REZNIKOV & A. NECHAEVA. *Byull. Nauch.-Tekh. Inform., Ministerstvo Geol. i Okhrany Nedr. S.S.S.R.*, 5:63 ('57); *Referat. Zhur. Geofiz.*, Abstr. No. 6498 ('58). Reaction of pptg. K in form of complex salt $K_2Na[Co(NO_2)_6]\cdot H_2O$ was used. 2 variations of method were proposed: volumetric, based on oxidation of complex salt with $KMnO_4$ in an acid medium, which gives good results for high K concns. (10 mg/l); and a turbidimetric variation based on comparison of intensity of cloudiness of coagulating ppt. of complex salt in water sample being studied and in a series of standard solns. A scale of acetylcelluloid can replace liquid standard scale. A field lab. was designed.—*CA*

Determination of the Oxidizability of Waters. R. BUYDENS & R. LEDENT. *Bul. Centre Belge Et. Documentation Eaux* (Liege), 38:238 ('57). Potassium permanganate, cerium sulfate, and potassium chromate methods of detg. oxygen demand of water are discussed individually, and suitability of the 3 reagents is compared. Potassium permanganate method gives reproducible results provided reagent is free from manganese dioxide, and under these conditions reagent is stable for several months. Sensitivity of cerium sulfate method is similar to that of potassium permanganate method, both methods giving results which increase with amt. of reagent used and with duration of boiling. Potassium chromate method is considered useful provided that only 0.25*N* chromate soln. and not 0.05*N* is used and that catalyst silver sulfate, sometimes used, is not added. 3 methods were used to examine pold. waters of 4 rivers (Seine, Orneau, Sambre, and Meuse) and water from distribution system at Brussels. Results are tabulated, expressed in milligrams of oxygen consumed per liter of water.—*WPA*

Determination of RaB, RaD, and ThB in Natural Waters. G. ALBERTI; C. BETTINALI; & F. SALVETTI. *Ann. Chim.* (Rome), 49:193 ('59). Radioactive isotopes of Pb are detd. by extn. from H_2O (contg. $Ba(NO_3)_2$, $Pb(NO_3)_2$, NH_4 citrate, NH_4OH, and $NH_2OH\cdot HCl$) with a soln. of dithizone in $CHCl_3$. $CHCl_3$ is evapd. and decay of the β-activity of residue examd. A similar expt. after 30 days gives Ra and ThX. Results, for water flowing from rock contg. U and Th, are: Rn 1.3×10^{-9}, ThB 1×10^{-10}, Ra $<10^{-9}$, ThX $<10^{-10}$, RaD $\sim10^{-10}$. It is shown that the rock (volcanic tuff) absorbs ThB when a Th soln. is filtered through it. —*CA*

Electrode System for Measuring Dissolved Oxygen. D. CARRITT & J. KANWISHER. *Anal. Chem.*, 31:5 (Jan. '59). A continuous measure of oxygen tension in fluid systems having wide range of properties found in natural fresh and sea waters, blood, and sewage has not been practicable. Polarographic devices using solid electrodes have had limited applicability to such systems because of electrode poisoning and high temp. coefficient. Device described overcomes major objections to previous oxygen-measuring instruments. It is temp.-compensated and, because of selectivity of membrane separating electrodes from environment, will function satisfactorily in a wide variety of aqueous systems. It can be used with continuous recording devices or as a completely portable instrument with readout on a microammeter.—*PHEA*

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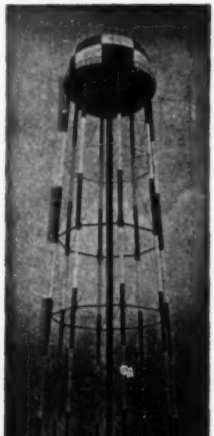

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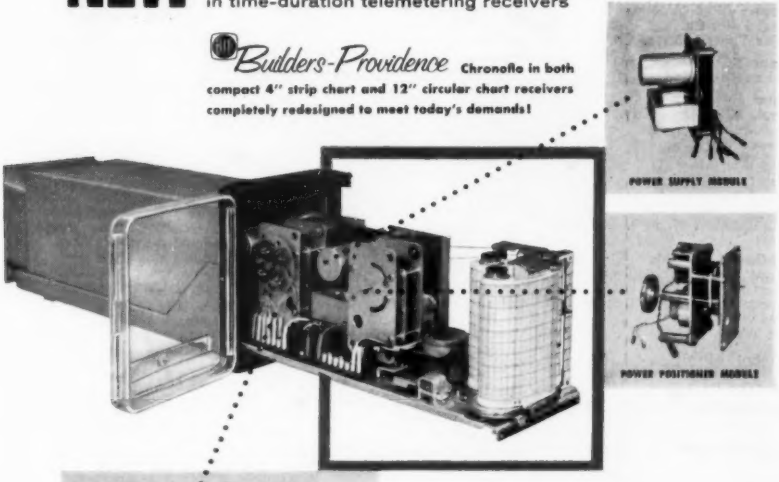

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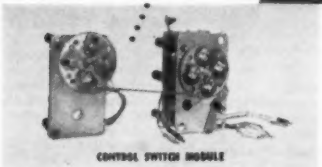

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- Non-Rising Stem
- Parallel Seats
- Bell Ends

FIG. 571X
- Inside Screw
- Non-Rising Stem
- Parallel Seats
- Mechanical Joint Ends

FIG. 561
- Inside Screw
- Non-Rising Stem
- Parallel Seats
- Flanged Ends

FIG. 566
- Outside-Screw-and-Yoke
- Rising Stem
- Parallel Seats
- Flanged Ends

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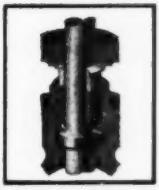

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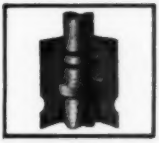

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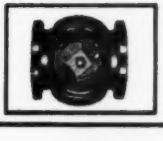

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A. P. Smith Mfg. Co.
E. H. Wachs Co.

Pipe Jointing Materials; see Jointing Materials

Pipe Locators; see Locators, Pipe

Plugs, Removable:
James B. Clow & Sons
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potassium Permanganate:
Carus Chemical Co.

Pressure Regulators:
Allis-Chalmers Mfg. Co.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
DeLaval Steam Turbine Co.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Chemical Feed:
B-I-F Industries, Inc.—Proportioneers
Precision Chemical Pump Corp.
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Layne & Bowler, Inc.
Peerless Pump Div.

Pumps, Diaphragm:
Dorr-Oliver Inc.
Wallace & Tiernan Inc.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Peerless Pump Div.
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Sump:
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Turbine:
DeLaval Steam Turbine Co.
Fiese & Firstenberger
Layne & Bowler, Inc.
Peerless Pump Div.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
B-I-F Industries, Inc.—Builders
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Reservoirs, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.

Sand Expansion Gages; see Gages

Sleeves: see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
Ludlow Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Infilco Inc.
Permutit Co.

Sodium Aluminate:
National Aluminate Corp.

Sodium Chloride:
International Salt Co., Inc.

Sodium Fluoride:
American Agricultural Chemical Co.

Sodium Hexametaphosphate:
Calgon Co.

Sodium Hypochlorite:
Jones Chemicals, Inc.
Wallace & Tiernan Inc.

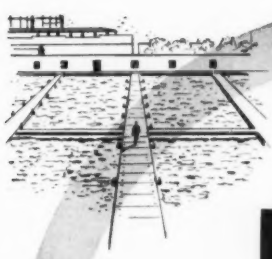
Sodium Silicate:
Philadelphia Quartz Co.

Sodium Silicofluoride:
American Agricultural Chemical Co.
Tennessee Corp.

Softeners:
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon Co.
General Filter Co.
International Salt Co., Inc.
National Aluminate Corp.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.



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Stops, Curb and Corporation:
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Hays Mfg. Co.
Mueller Co.

Storage Tanks: see Tanks

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R. D. Wood Co.

Surface Wash Equipment:
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Permutit Co.

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B-I-F Industries, Inc.—Omega
B-I-F Industries, Inc.—Proportioneers
Wallace & Tiernan Inc.

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National Tank Maintenance Corp.
Taylor Iron Works

Tanks, Prestressed Concrete:
Preload Co., Inc.

Tanks, Steel:
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Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Morgan Steel Products, Inc.
Pittsburgh-Des Moines Steel Co.
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Tapping-Drilling Machines:
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Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.

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B-I-F Industries, Inc.—Proportioneers
General Filter Co.
Graver Water Conditioning Co.
Industrial Chemical Sales Div.
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Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.

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DeLaval Steam Turbine Co.

Turbines, Water:
DeLaval Steam Turbine Co.

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Ludlow Valve Mfg. Co., Inc.
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Mueller Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

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A. P. Smith Mfg. Co.

Valves, Altitude:
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Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.

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B-I-F Industries, Inc.—Builders
Chapman Valve Mfg. Co.
James B. Clow & Sons
DeZurik Corp.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Henry Pratt Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey-Sparling Meter Co.

Valves, Electrically Operated:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
A. P. Smith Mfg. Co.

Valves, Float:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Henry Pratt Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Henry Pratt Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Large Diameter:
Allis-Chalmers Mfg. Co., Hydraulic Div.
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Henry Pratt Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Regulating:
Allis-Chalmers Mfg. Co., Hydraulic Div.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Henry Pratt Co.
Ross Valve Mfg. Co.

Valves, Swing Check:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
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Venturi Tubes:
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Simplex Valve & Meter Co.

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Inertol Co., Inc.
Koppers Co., Inc.
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Water Softening Plants; see Softeners

Water Supply Contractors:
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Water Testing Apparatus:
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Water Treatment Plants:
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Chain Belt Co.
Chicago Bridge & Iron Co.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

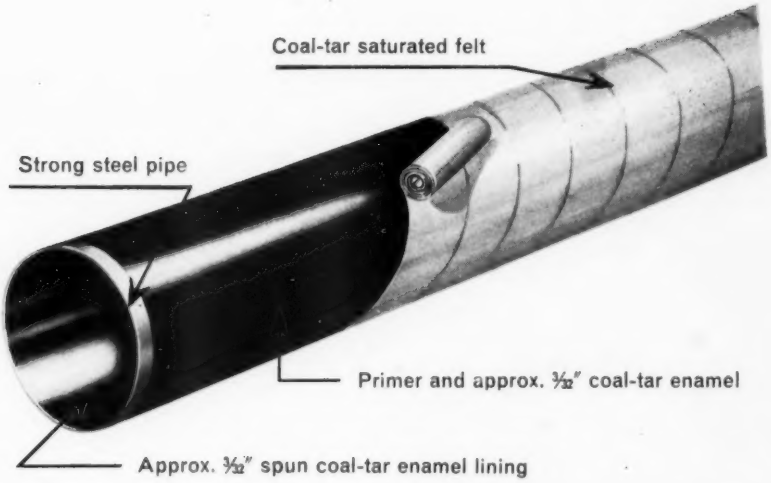
Well Drilling Contractors:
Layne & Bowler, Inc.

Well Reconditioning and Formation Testing:
Halliburton Oil Well Cementing Co.
Layne & Bowler, Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite: see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by AWWA Associate Members appears in the 1959 AWWA Directory.



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When you invest in a water line, it makes good economic sense to play safe—specify steel pipe, lined and coated with coal-tar enamel.

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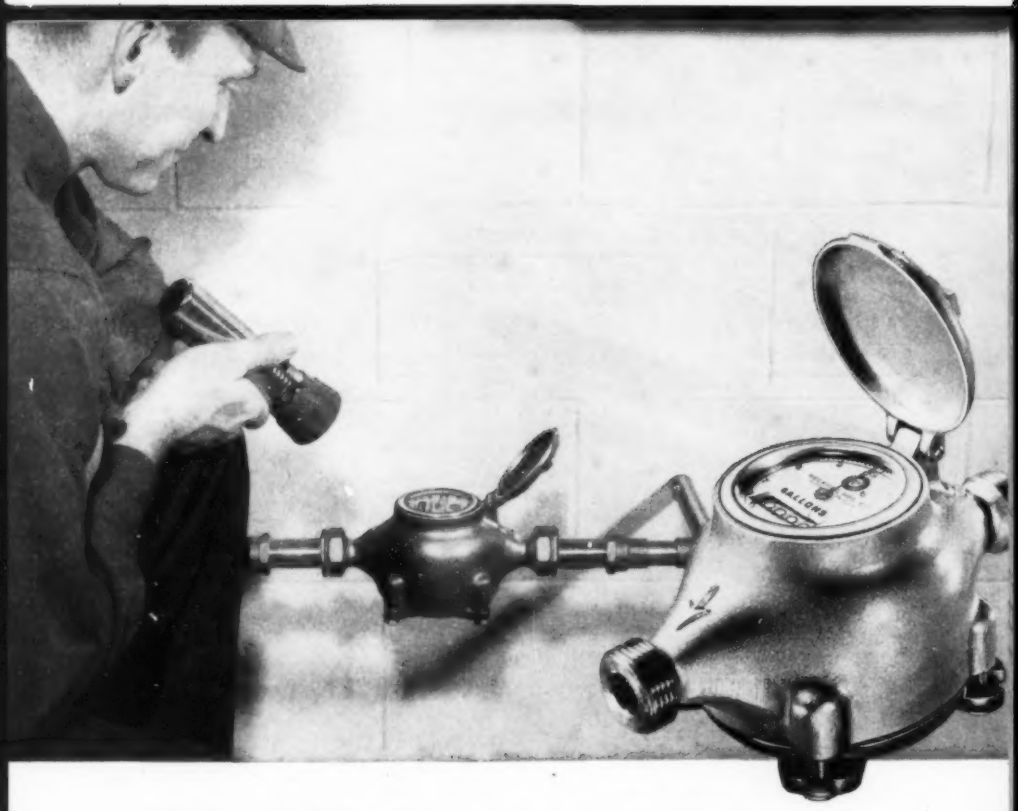

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This construction permits the sealed register to be positioned high and dry above the flowing stream. Condensation and dirt, the bane of the meter reader, just can't occur. In fact, this meter will run under water without harm.

The sealed register assembly contains all the gearing both for register operation and for intermediate reduction. Gears and pinions are of the free running instrument type. They offer only token resistance to the measuring element and so improve the accuracy and response of the meter especially at low flows.

All this, plus a simplified design in which only two parts operate in water means less wear, less chance for corrosive attack and of course easier maintenance at lower cost. Get facts now. Write to Rockwell Manufacturing Company, Pittsburgh 8, Pa.

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Transite's maintained carrying capacity keeps pumping costs low. Its strength, durability and corrosion-resistance cut maintenance—provide years of trouble-free service. Further savings are effected by the sure coupling method that assures a tight seal at every joint. Add to this Transite's natural interior smoothness and you see why it helps reduce community water system costs. Let us send you "Facts and Data for Engineers" book. Write Johns-Manville, Box 14 (JA-2), New York 16, N. Y.

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